CONCRETE
AND
CONSTRUCTIONAL ENGINEERING

DECEMBER, 1950. **Vol. XLV, No. 12**

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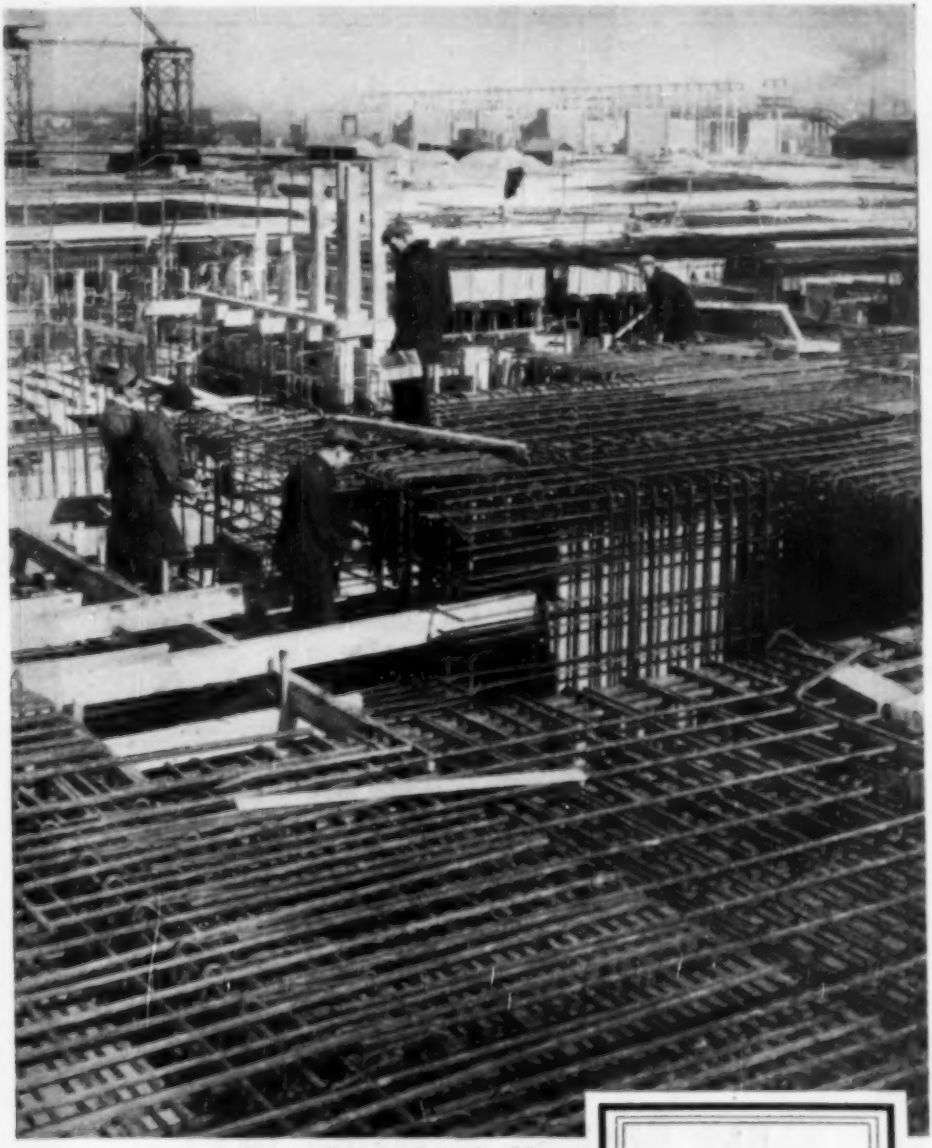

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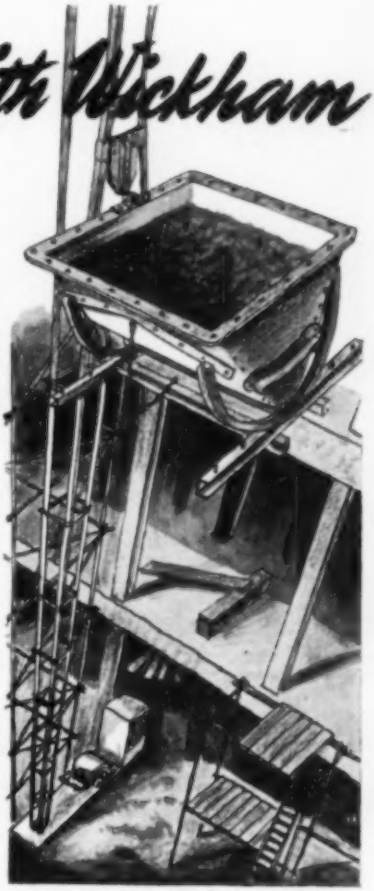

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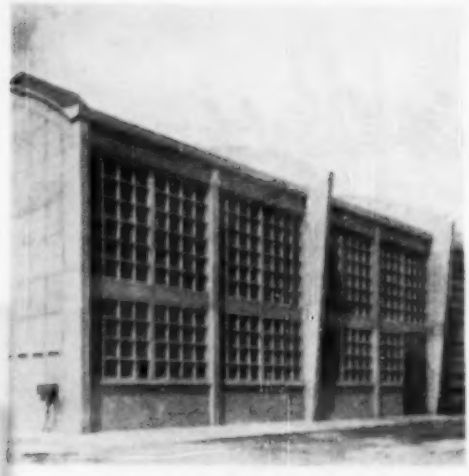

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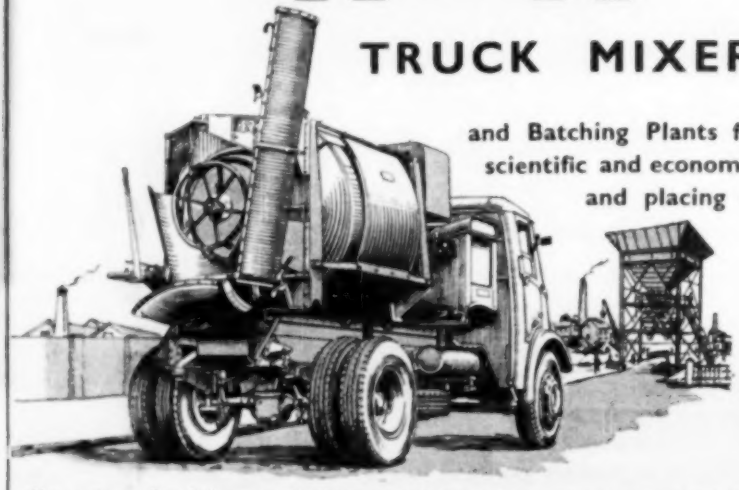

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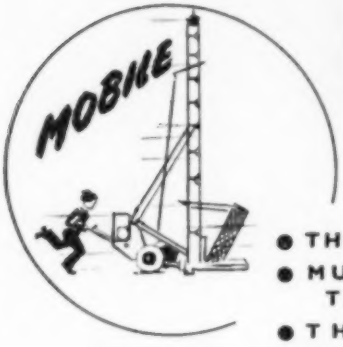

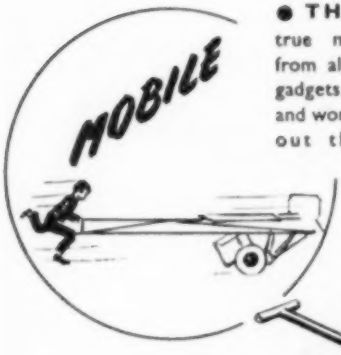

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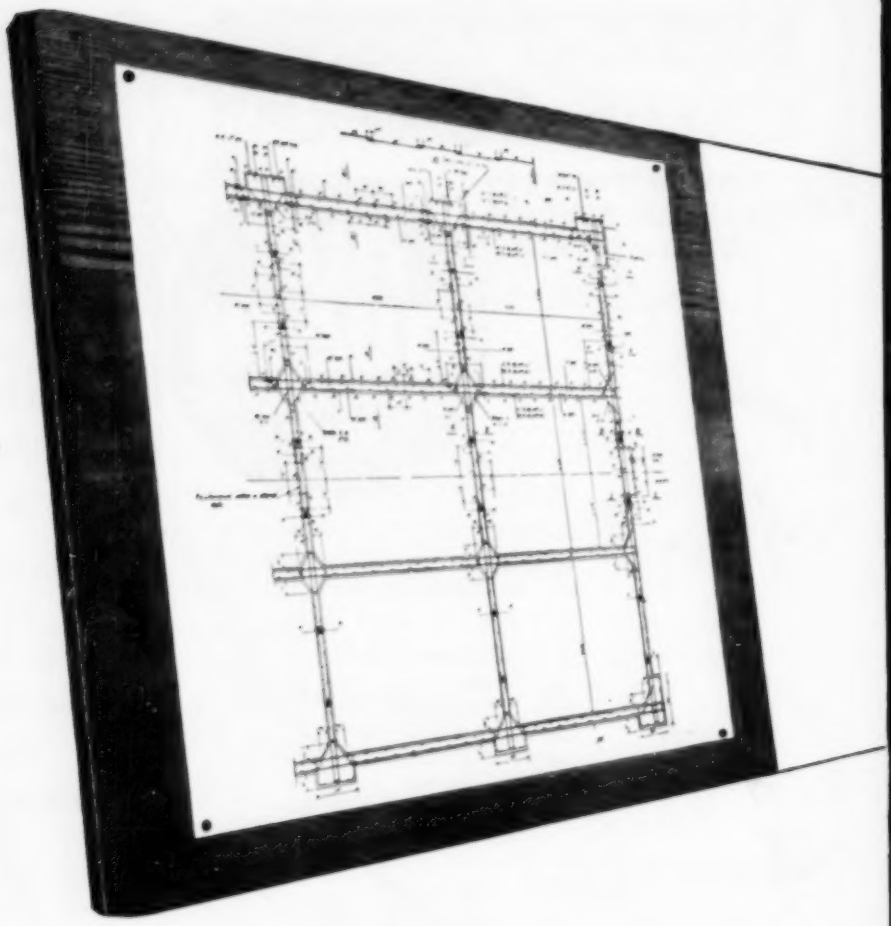

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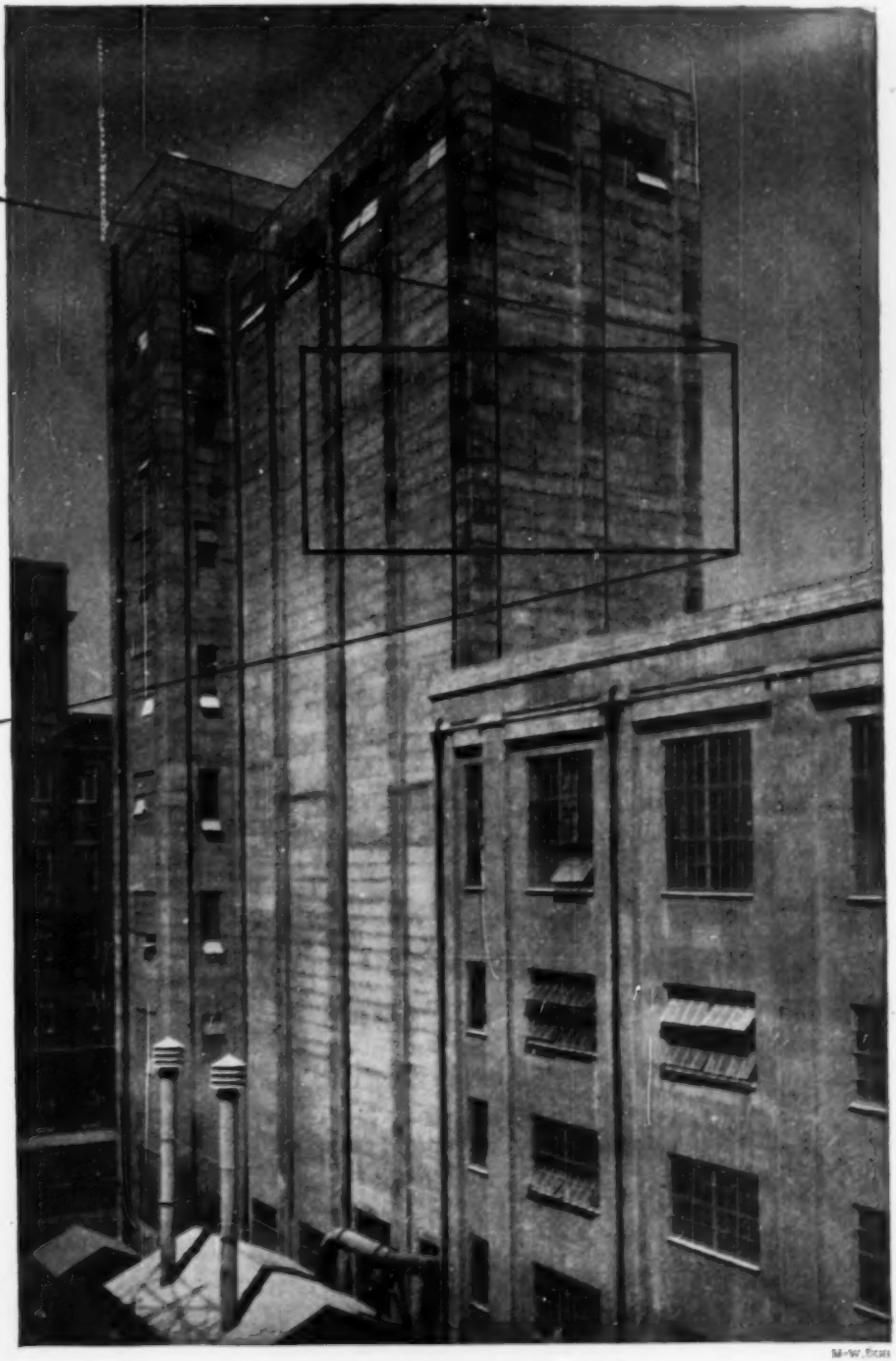

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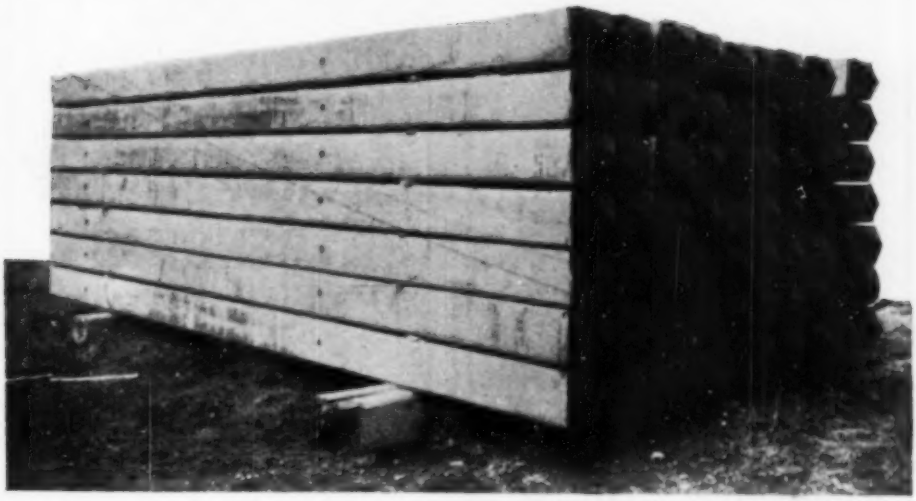

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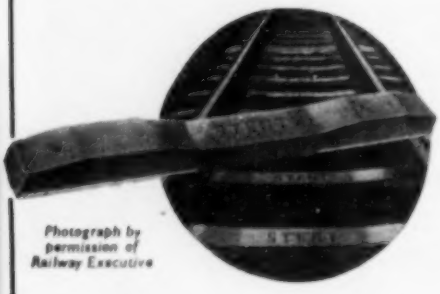

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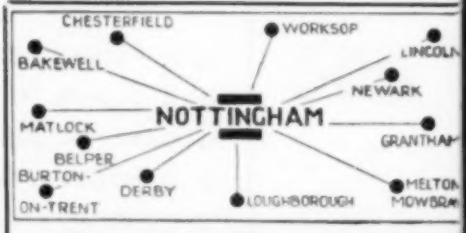

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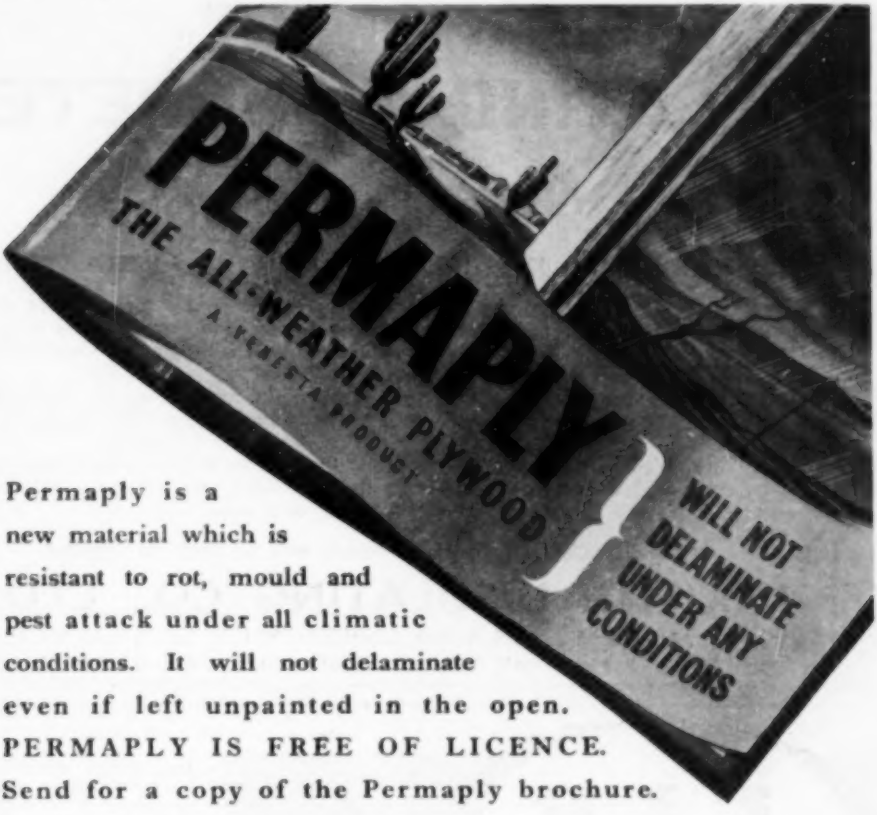

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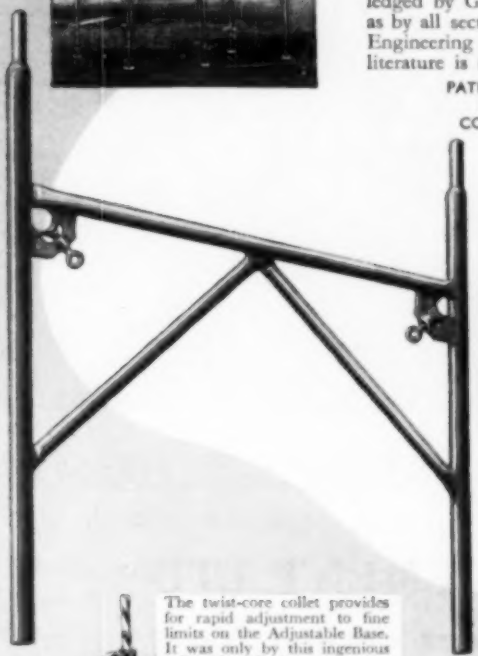

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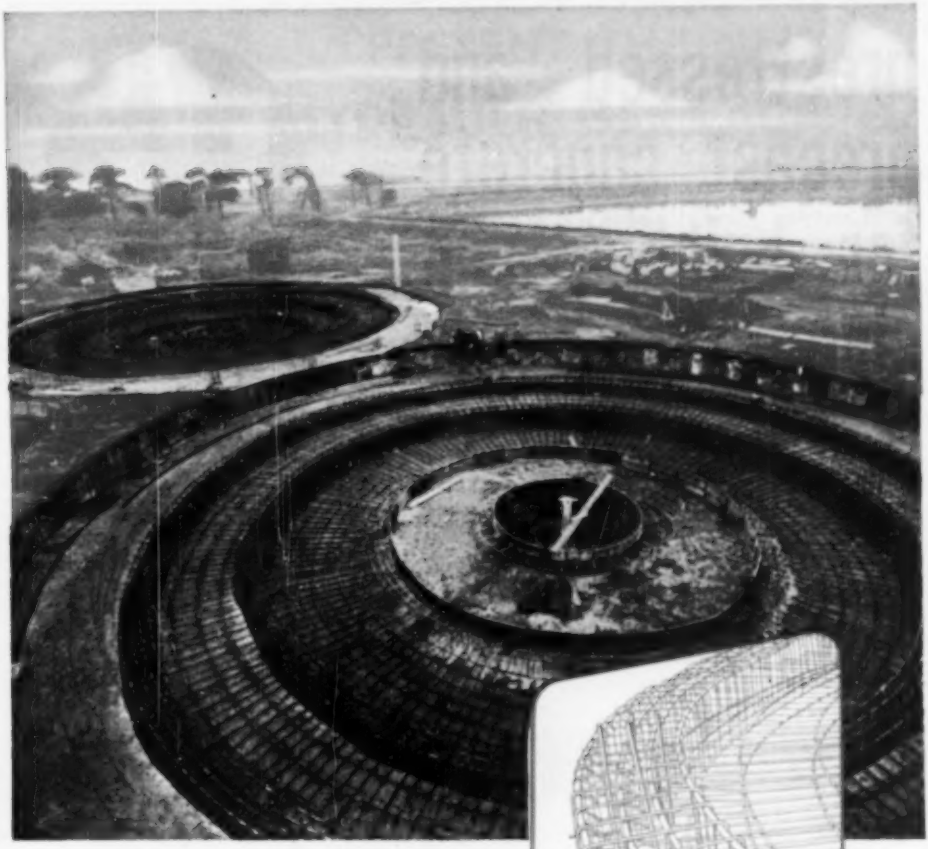

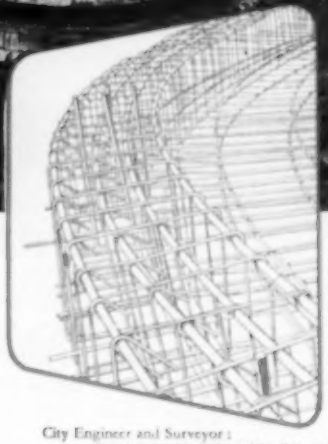

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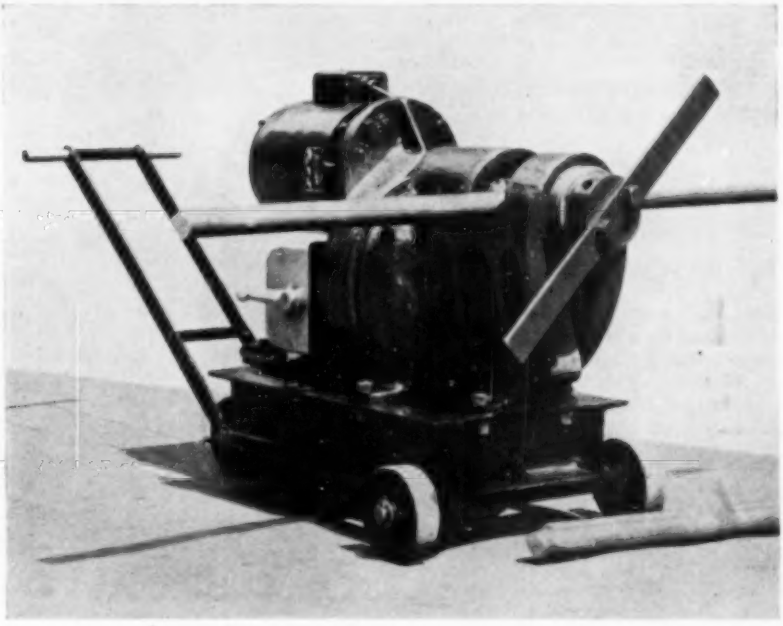

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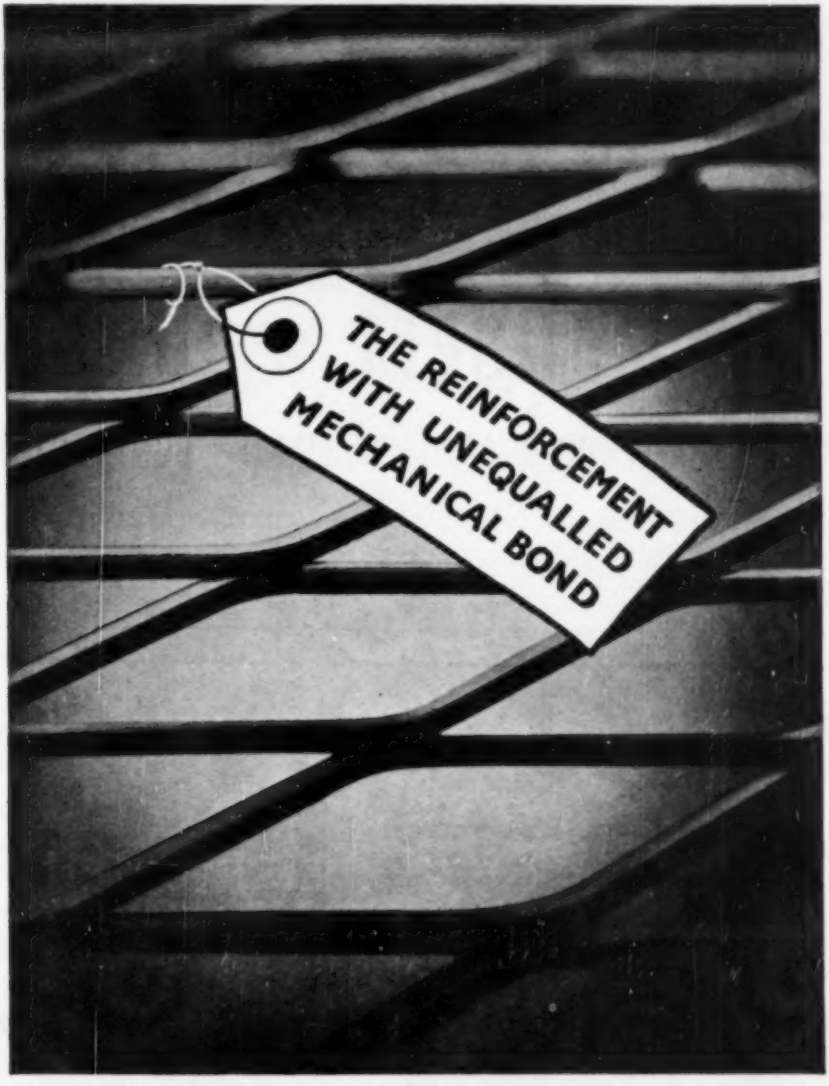

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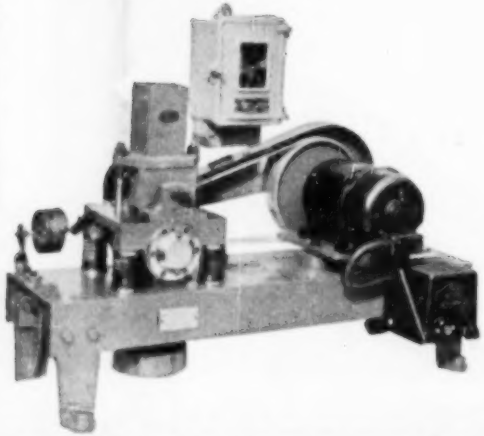

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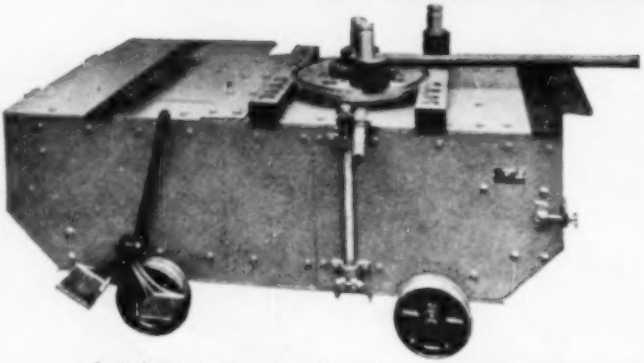

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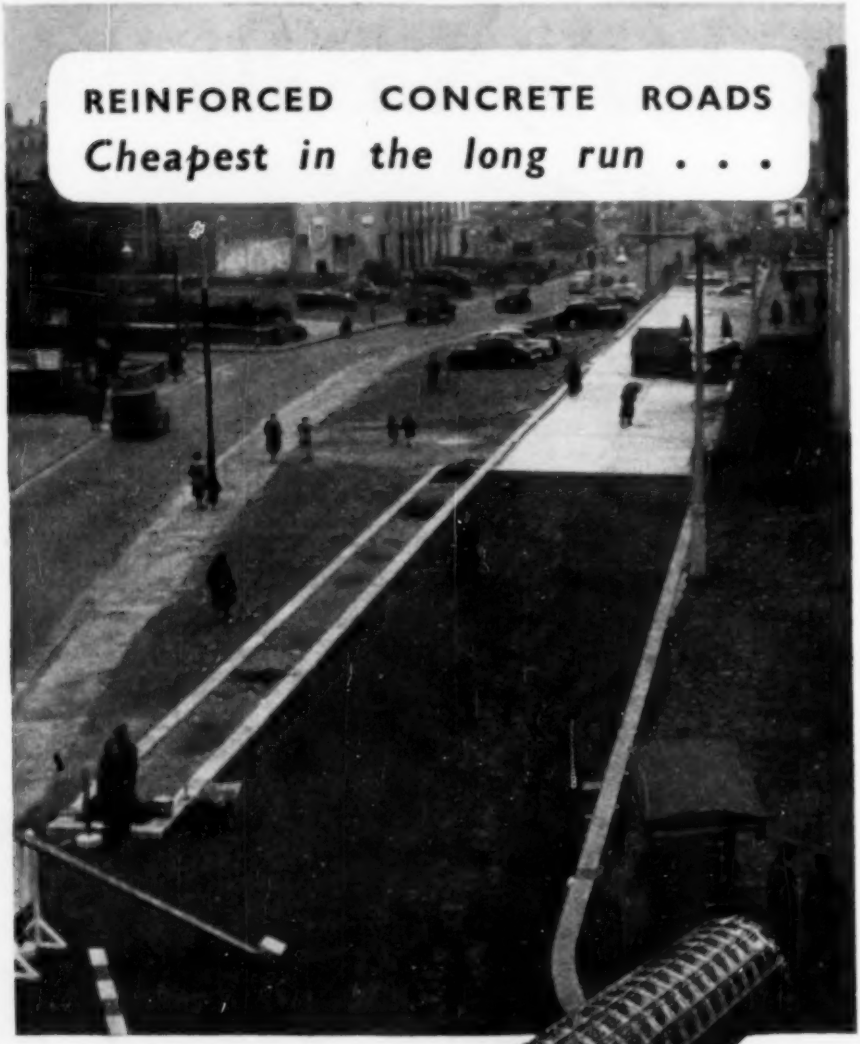

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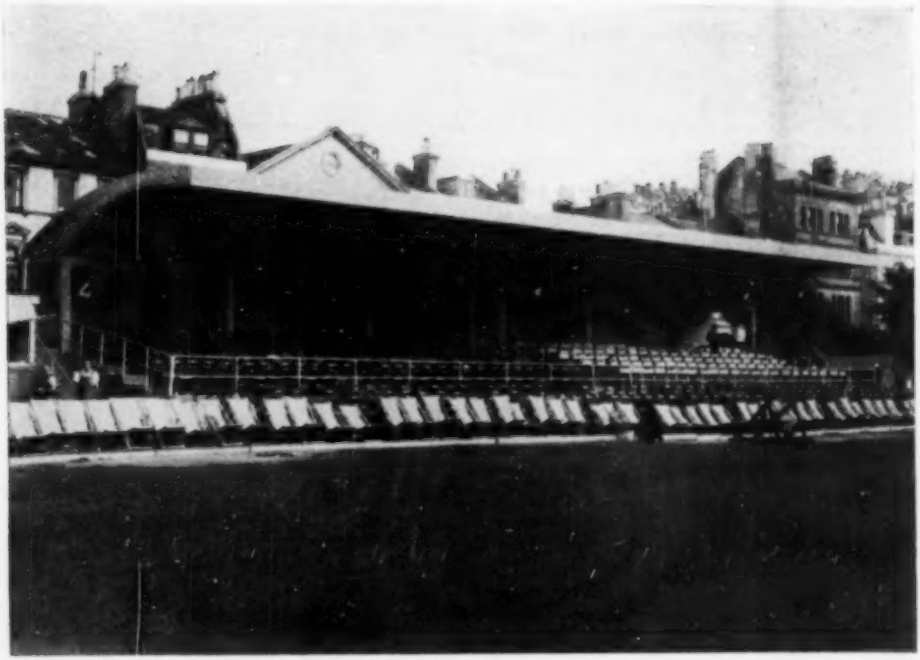

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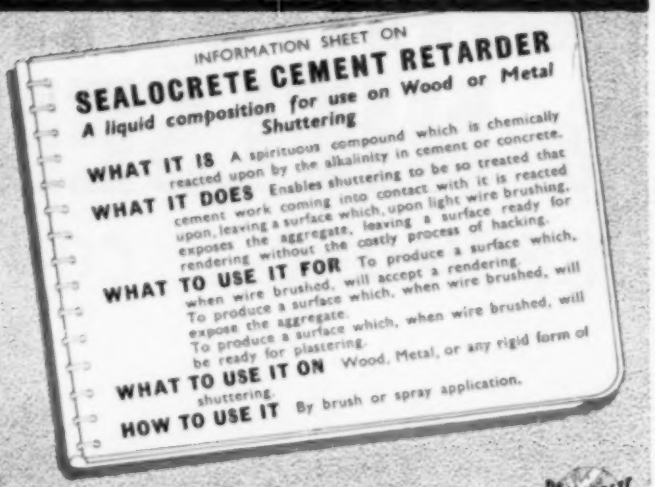
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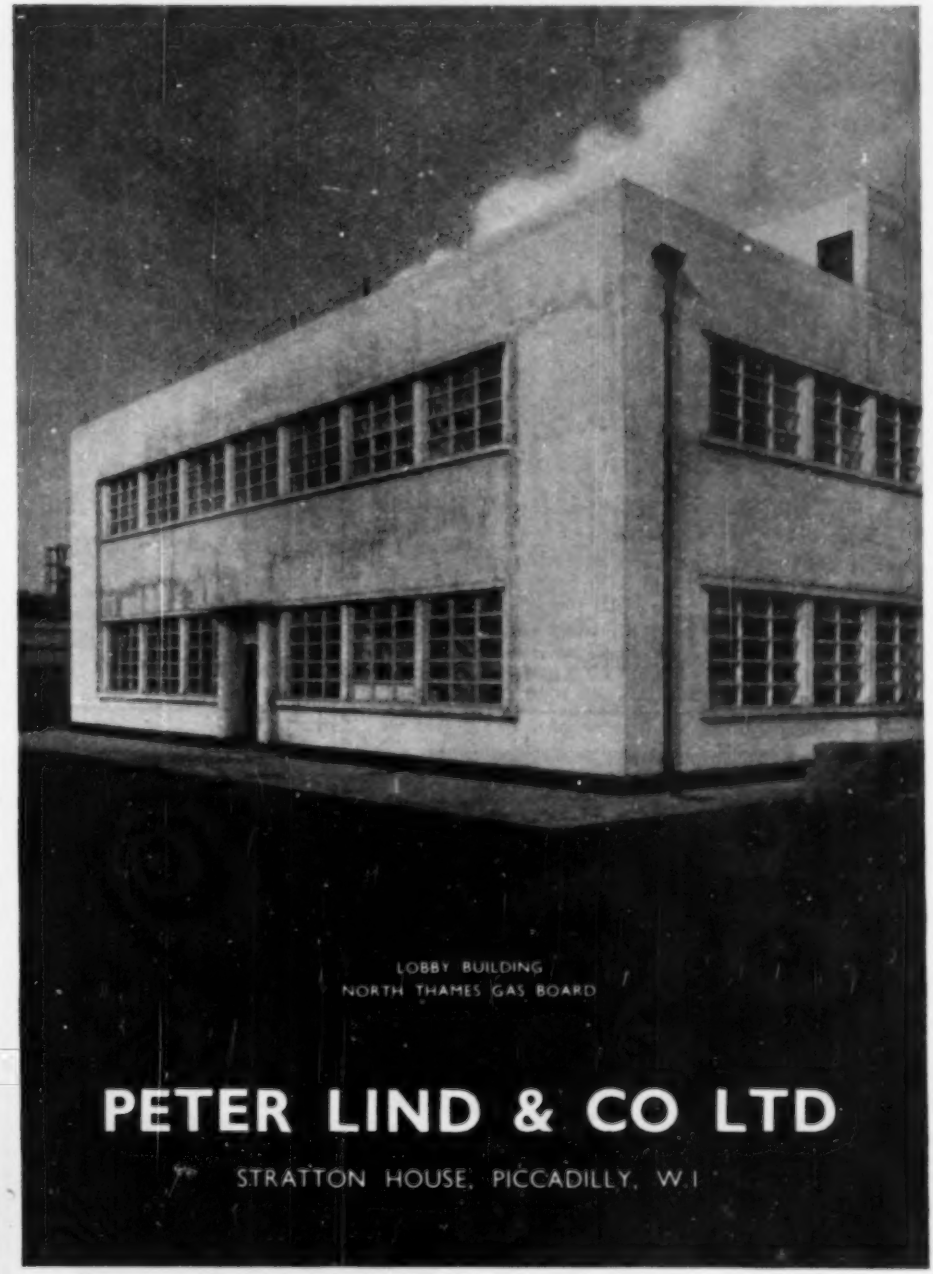
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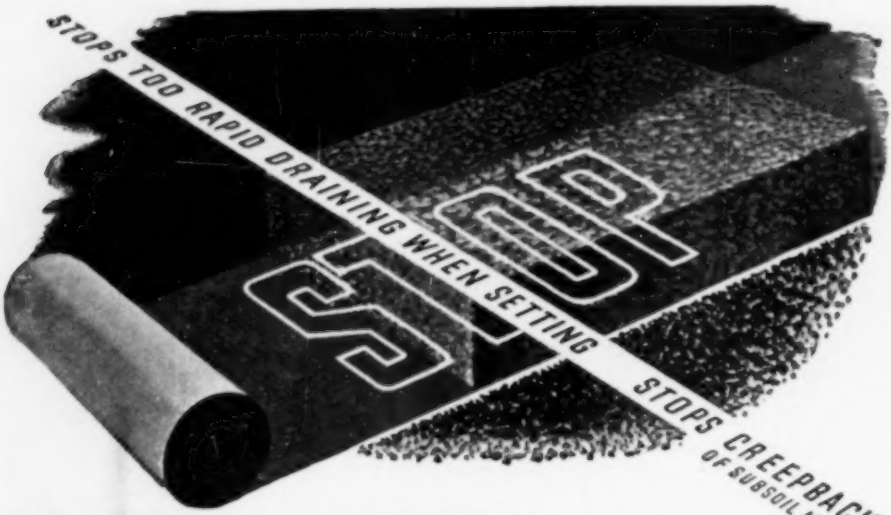

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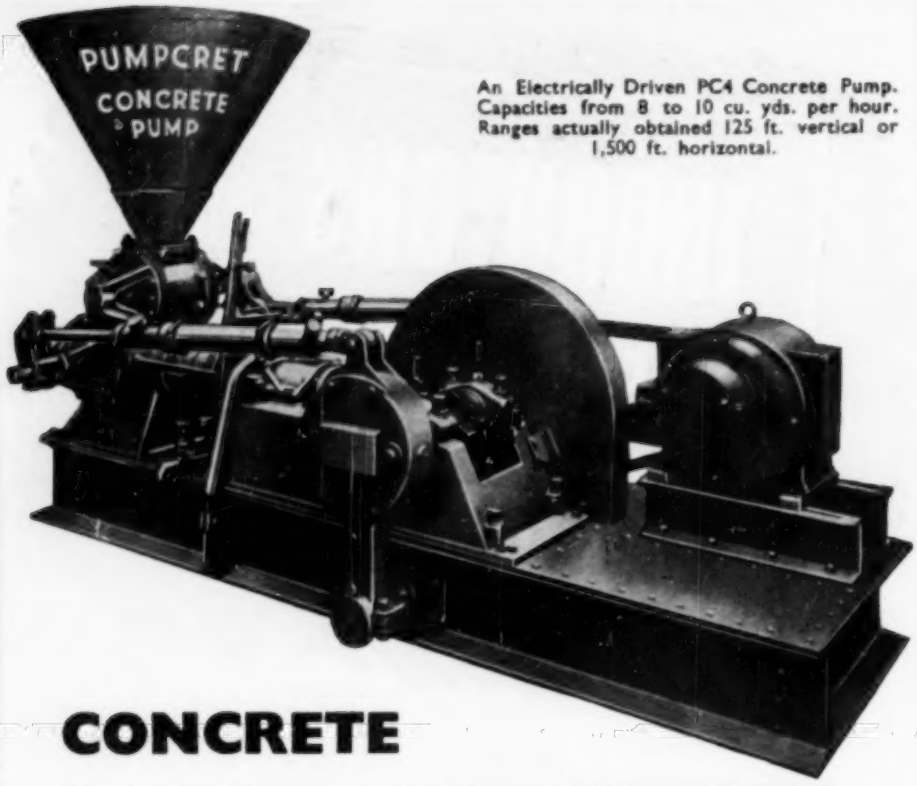

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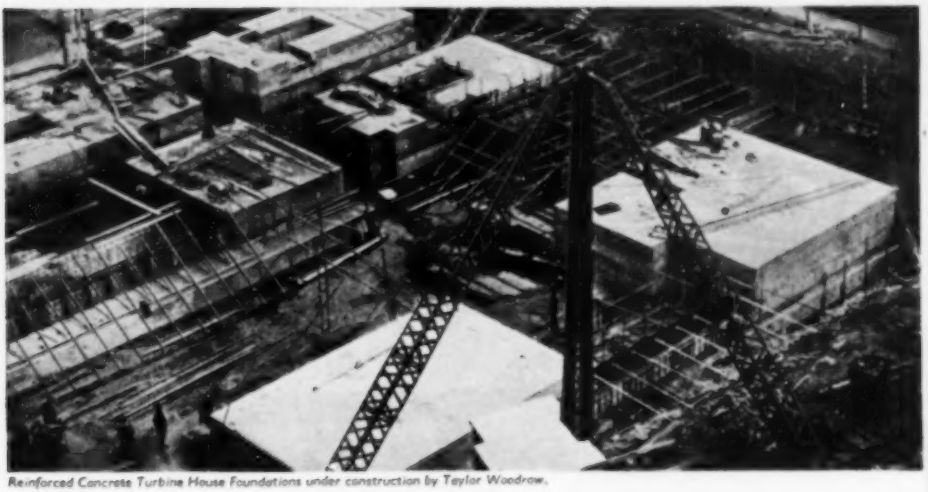

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CONCRETE
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Volume XLV. No. 12. LONDON, DECEMBER, 1950

EDITORIAL NOTES

Developments in Prestressed Concrete.

A FEW years ago the theory applied to prestressed concrete was comparatively simple, but recently more theories have been devised to explain the behaviour of compressed concrete beams under working load and at failure. That a reliable theory may not be so simple as at first thought may be realised from a consideration of some of the properties of prestressed concrete that have been established by tests. A beam may be loaded until it is cracked; upon reducing the load the cracks apparently close completely and the beam continues to behave under smaller loads as if it were a homogeneous material. The ultimate load-carrying capacity of a prestressed concrete beam may not depend upon the amount of the initial stretching force in the wires provided that the wires are bonded to the concrete and that the amount of wire is such that the steel yields before the concrete crushes. The deflection, however, is greater the smaller the stretching force. This ability of a prestressed concrete beam to carry the same load, irrespective of the amount of initial tension in the wires, so long as the concrete is not crushed before the wires yield, may be an answer to the question of what happens if in course of time the prestress in the wires is seriously reduced.

What happens at failure is important, because the factor of safety may bear little relation to the stresses at working load. For example, the stress in the wires at working load may be less than that when the wires are first stretched, and may be two-thirds of the tensile strength of the wires. Opinions differ regarding what happens to the compression in the concrete at failure. Some consider that there is no longer any precompression in the concrete in the lower half of the beam because it cannot be transferred across the cracks in the concrete. Others consider that the precompression is still fully effective but is transferred through the upper and compressed part of the beam. May not this problem be connected with the deflection of the beam and the consequent reduction in the equivalent eccentricity at midspan?

These matters are no doubt resolved in one way or another to the satisfaction of exponents of different ideas. There is, however, a major difference of opinion about whether tensile stress in the concrete may be, or must not be, permitted under the working load. If tensile stresses are permitted, the question arises whether or not the tensile stresses may or may not be sufficiently large to cause the concrete to crack under loads not exceeding the working load. It must not be overlooked that a beam designed not to crack under working load may be

accidentally cracked, say, by mishandling or because of restrained shrinkage; the condition of tensile stresses without cracks is then apparently unobtainable, although the phenomenon that cracked prestressed concrete may behave as a homogeneous material has a bearing on the matter.

A matter that should have serious consideration is the possibility of producing an acceptable notation for use throughout the world. Notations are developed in a haphazard manner at the whim of committees and individuals. An original investigator uses a notation that seems to him to be reasonable, and he may be excused if he thinks that the invention is more important than the method of explaining it. Others writing on the same theory use what they consider to be a better notation, until the time arrives, as it has now done in the case of prestressed concrete, when numerous notations are used by different investigators and authors and the same symbols have different meanings in different books and papers even when they are published in the same country and sometimes by the same publishers. This makes the study of a subject unnecessarily laborious, and leads to the same confusion of thought that arises when the advocates of opposing political faiths use the same words to mean different things. We have stated our view that the subject of prestressed concrete is not sufficiently developed to warrant the issue of a code of recommended practice, but it is not too soon for the adoption of an international standard notation so that in the use of symbols at least we shall all write the same language. Indeed, if this is not done soon it will be too late to do so, for it is difficult to change things that have become established either in different countries or in books that have become looked upon as standard works.

In this journal for September last a method was described of designing prestressed concrete beams directly from the assumed properties of the materials and the elementary principles of equilibrium. Stress-strain curves are applied to convert strains to stresses, instead of using an assumed value of the elastic modular ratio. The use of the common formula, in which the moment of resistance is equated to the least product of the section-modulus and stress, is avoided. The conditions at the time of transfer of the prestressing force to the concrete and at failure of the beam only are considered to be critical. Such departures from the ordinary basis of design have caused criticism not so much of the correctness of the method as of the data assumed.

Forsaking the section-modulus naturally evokes comment, but the view of the author of the method proposed is that the ordinary basis of design does not take into account the non-rectilinear stress-strain relationship of concrete and cannot be applied to sections that have cracked. The assumption of a stress-strain curve for concrete in particular has been criticised because such curves depend on the size, shape, age, history and strength of the concrete member. Until concrete has been made and tested, the data necessary to draw such a curve are not available, whereas it is suggested that tests on prestressed concrete beams have established sound values of the elastic modular ratio. The author contends that the assumption of a single modular ratio is the assumption of stress-strain curves for concrete and steel without knowledge of probable variations at different stresses, and that the assumption (because it cannot be obtained practicably) of a short-term curve for concrete errs on the side of safety as it gives a high elastic modulus. The short-term curve in the article referred to was drawn on the

assumption that the strain at any stress is one-quarter of the strain of a specimen loaded so slowly that perceptible creep has ceased when the strain is measured. It is no more necessary to use the published curves, if a more accurate curve is available, than it is to assume invariably a modular ratio of, say, five, if a more accurate value is obtainable.

The assumption that the stress in the wires at failure of a beam is the 0·2 per cent. proof stress is criticised on the ground that tests show that the stress at failure is approximately the tensile strength if the wires are grouted in the beam. A greater stress at failure leads to a reduction in the amount of steel, but the author states that his assumption results in a compromise between saving steel and reducing the weight of the beam. Reduction in the weight of the beam, which is economically advantageous even if steel is not scarce, necessitates small stresses in the concrete at transfer, and to obtain such small stresses without increasing the size of the beam it is necessary that the increase in stress in the wires from transfer to failure should be as large as practicable, a result obtained by using the steel at stresses where the elastic modulus is high.

While these matters are being discussed, the practice of prestressed concrete construction develops. Although, quite rightly at this stage, no standard codes of practice have yet been published in this country, there is a Polish standard specification for prestressed concrete with bonded wires, and provisional recommendations for the design of prestressed concrete have been published in Germany. A supplement to the code for liquid containers issued by the Institution of Civil Engineers gives recommendations for the use of prestressed concrete for such structures. The size of structures built in prestressed concrete increases. Monolithic prestressed concrete building frames and continuous-girder bridges are among the latest developments. There are now about a hundred structures in Great Britain in which prestressed concrete is used either for the entire structure or for some part such as precast beams for floors. There is at least one building in London with prestressed concrete beams designed in accordance with the requirements of the London County Council. The largest span for a bridge in this country is as yet only 80 ft., which is small compared with spans exceeding 200 ft. of some structures abroad. In the north of England there are prestressed concrete tanks of 106 ft. diameter and 41 ft. deep.

Much publicity continues to be given to achievements in prestressed concrete. Courses giving theoretical and practical instruction are available. Wire suitable for the cables in prestressed concrete structures is more plentiful. New apparatus, some of which are ingenious and effective, are being devised. New patents are continually being added to the already long list of inventions. New companies have been formed to exploit this method of construction and it is said that, as a result of the consequent competition and extended experience, the cost of prestressed concrete construction is being substantially reduced. It seems that prestressed concrete is becoming established as an economical method of construction, and that experience is now sufficiently extensive to prove that prestressed concrete is a sound means of solving certain structural problems. It seems, also, that its shortcomings are now sufficiently realised to ensure that the method is not used in unsuitable cases.

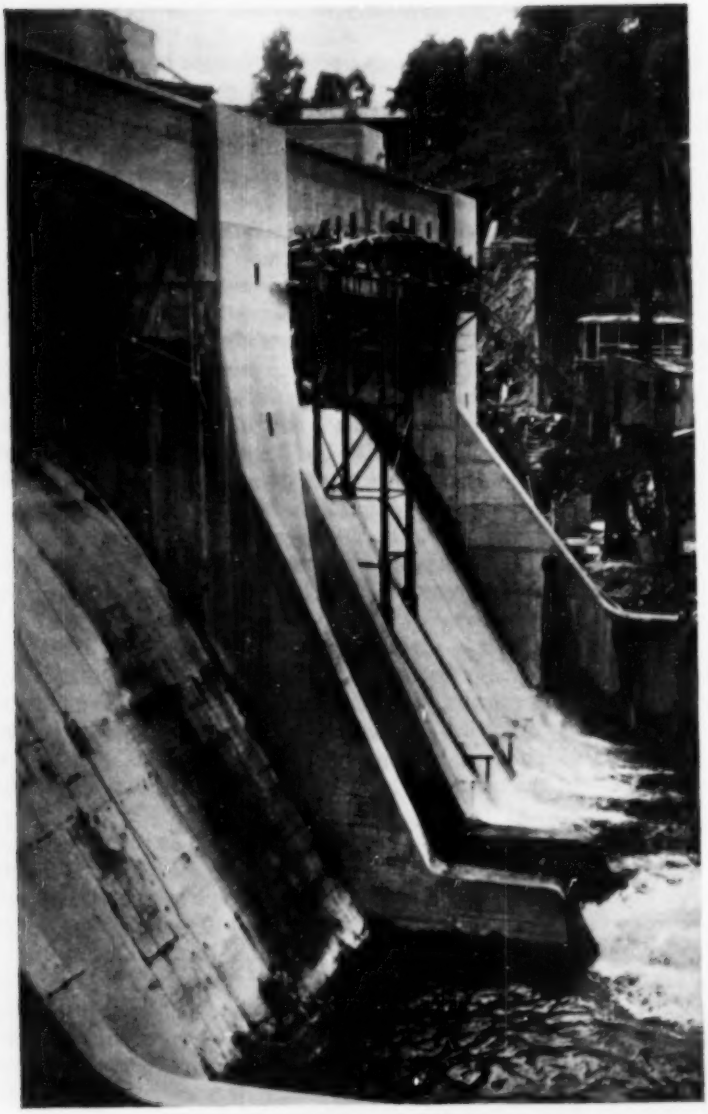


Fig. 6.—Clunie Dam: Spillway completed.
(*See facing page.*)

Scottish Hydro-electric Works.

THE accompanying illustrations show some of the structures in two of the hydro-electric works of the North of Scotland Hydro-electric Board, which are now completed or nearly completed.

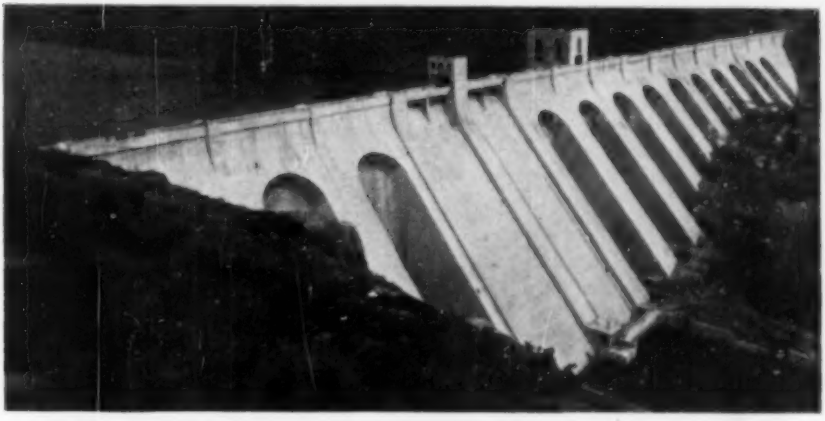


Fig. 1.—Loch Sloy Dam.

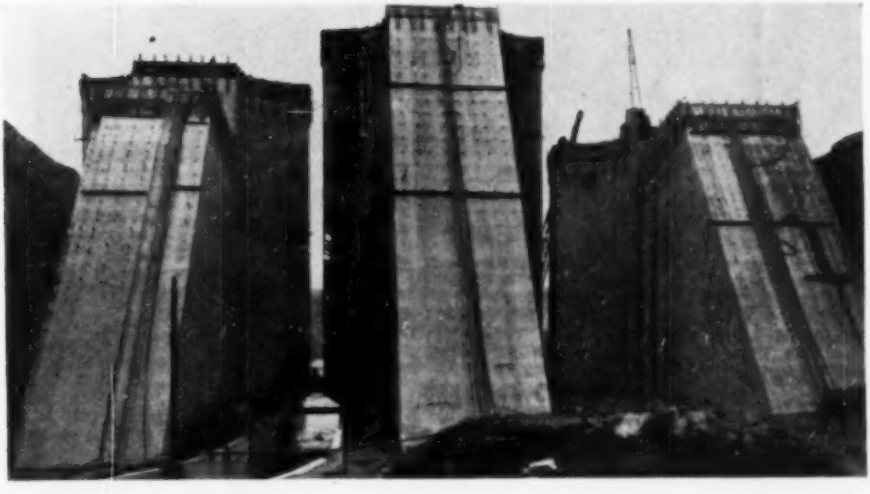


Fig. 2.—Loch Sloy Dam: Buttresses.

Loch Sloy Dam.

The construction of the Loch Sloy dam and tunnel and the Inveruglas generating station was described in this journal for March 1947 and December 1948. The dam (*Fig.* 1) is a mass-buttress concrete structure 1160 ft. long. It has a maximum height of 165 ft., and a volume of about 200,000 cu. yd. The central part of each buttress is 26 ft. wide, and this and the curved flanks were constructed first as separate blocks (*Fig.* 2) leaving inter-

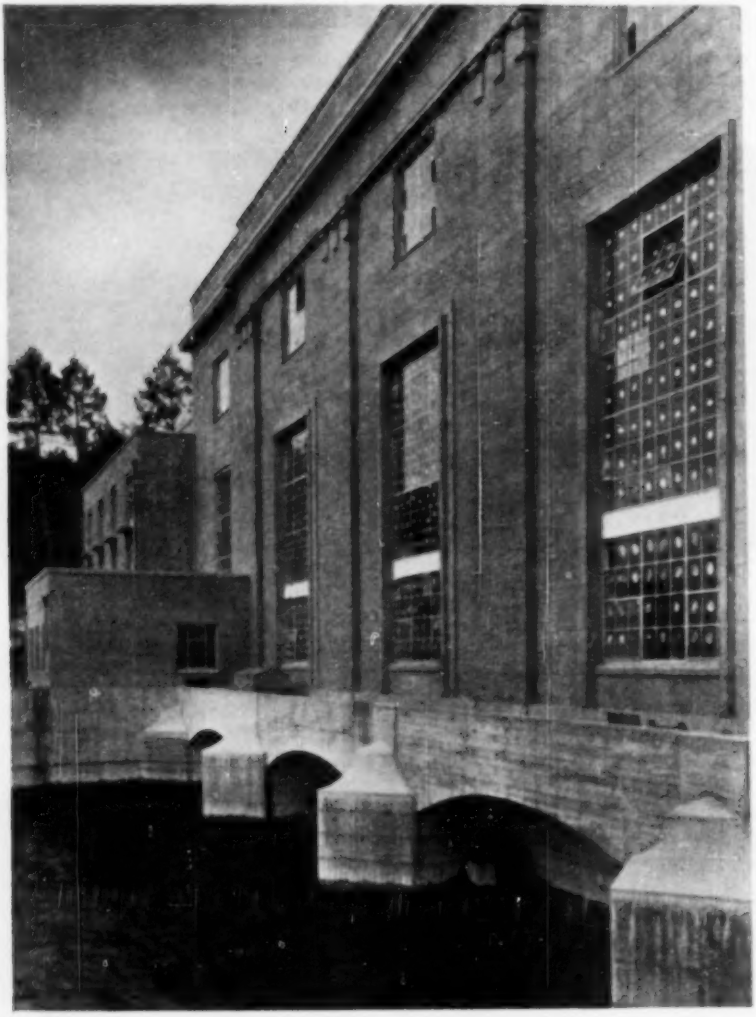


Fig. 3.—Inveruglas Generating Station.

mediate construction slots which were filled later. The continuous wall over the arched heads of the buttresses carries a road. The two spillways are separated by a projecting buttress on the head of which is the valve house.

Inveruglas Generating Station.

Water stored behind the dam at Loch Sloy is conducted to the generating station through the main tunnel, which is 2 miles long and of horse-shoe cross section (equivalent to 15 ft. 4 in. in diameter), and is lined mainly with concrete. Steel pipes lead from the tunnel to the generating station (*Fig.* 3) at Inveruglas. The station contains four vertical turbo-alternators having a combined capacity of 130,000 kw. The station was put into operation in October 1950.

The walls comprise 9½ in. of reinforced concrete cast in situ in steel shuttering.

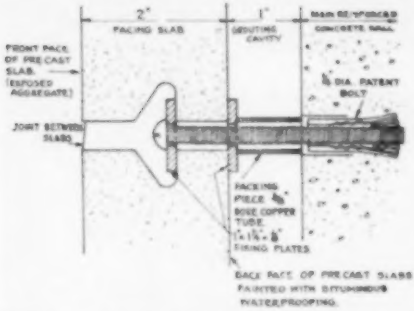


Fig. 4.—Inveruglas Generating Station: Fixing Facing Slabs to Concrete Walls.

The external face is covered with precast concrete slabs erected after completion of the main wall and separated therefrom by a 1-in. space filled with grout, the slabs being secured to the wall by bolts on which copper distance pieces are threaded as shown in *Fig.* 4. The slabs are in courses of 24 in. and 13½ in. alternately and are 2 in. thick. The backing concrete is 1½ in. thick, the mixture being 1 part of ordinary Portland cement to 3½ parts of crushed stone. The facing concrete is ¾ in. thick and is also a 1 : 3½ mixture, the aggregate being crushed granite of three types, namely ⅜-in. to ⅛-in. grey granite, ⅛-in. to ¼-in. blue granite, and ¼-in. down comprising equal parts of grey and pink granite. In the dressings surrounding windows and elsewhere the aggregate is grey granite of ¼ in. maximum size. The cement in the facing concrete is 85 per cent. of ordinary Portland cement and 15 per cent. of buff-coloured cement. The facing was given repeated scrubbings to expose the aggregates. The backs of the slabs were painted with a bituminous waterproofing compound.

Tummel-Garry Works.

The construction of the works at Clunie and Pitlochry, which comprise the first

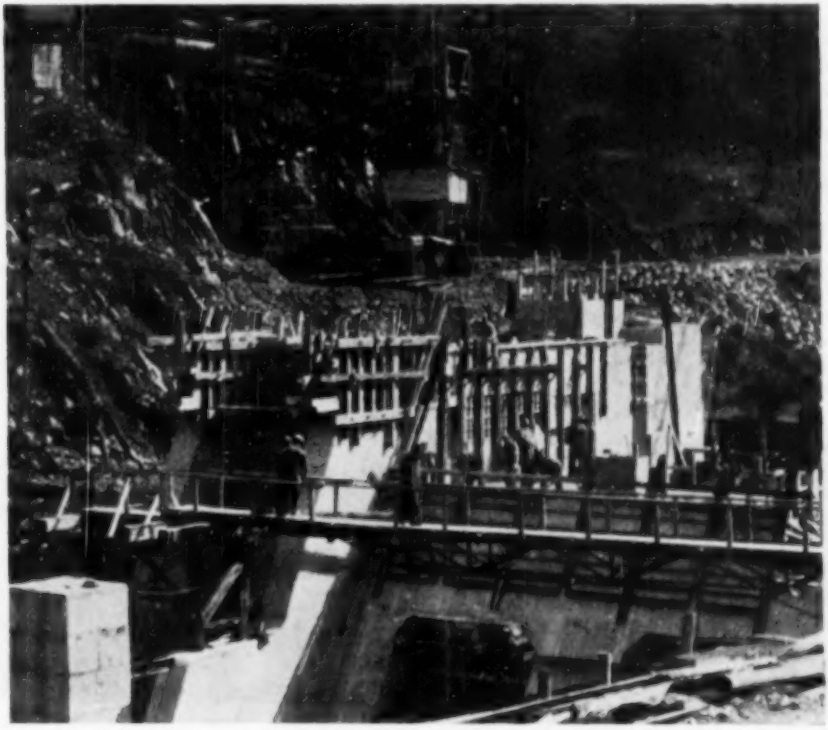


Fig. 5.—Clunie Dam in Course of Construction: Downstream Face.

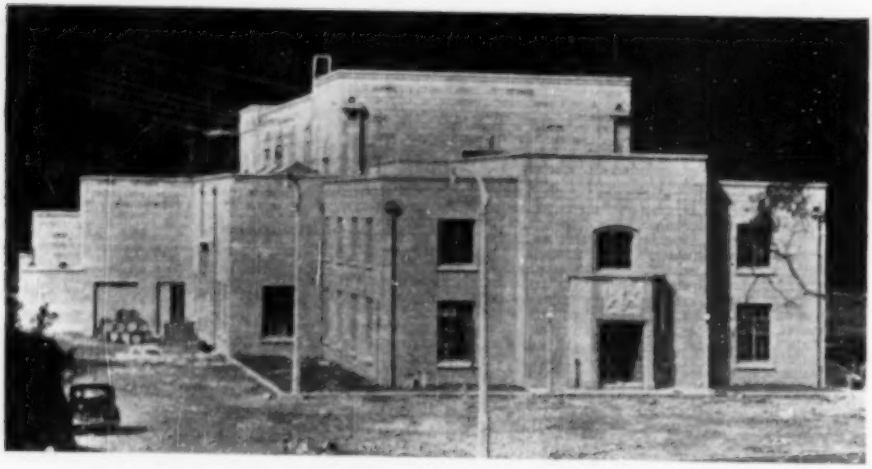


Fig. 7.—Clunie Generating Station.

part of the Tummel-Garry scheme, were described in this journal for January and February 1949. The dam at Clunie (*Fig.* 6, page 418), now completed, is a gravity structure 380 ft. long, has a greatest width and height of 90 ft. and 70 ft. respectively, and contains about 37,000 cu. yd. of concrete. Another view of the downstream face at an early stage of construction is given in *Fig.* 5, which shows the temporary access bridge, shuttering, and, in the background, the plant for handling the rock excavated from the tunnel.

The tunnel which conducts the water from behind the dam to Clunie generating station is about two miles long. It is of horse-shoe cross section having an equivalent diameter of 23 ft., and is the largest water tunnel in Britain. Some notes on the construction of the tunnel, which was driven to the full size from two faces simultaneously, were given in this journal for January 1949. The tunnel is lined with concrete, the walls and crown being lined at the same time using a travelling shutter 100 ft. long. The coarse aggregate for the lining is selected crushed schist excavated from the tunnel. The fine aggregate is natural sand. The batched materials were transported in skips to three ¾-cu. yd. concrete mixers mounted on a carriage immediately behind the shutter, and the concrete was placed by two 6-in. pumps.

The installed capacity of the generating station at Clunie (*Fig.* 7) is about 60,000 kw., and is now in operation. The shuttering for the inlet to the draught tube of one of the three turbo-alternators is shown in *Fig.* 9. The reinforced concrete walls of the building were cast in 2-ft. lifts

Fig. 8.—Pitlochry Dam and Generating Station.

between steel shuttering on the inside and precast concrete facing slabs on the outside. The exposed granite-aggregate slabs are similar to those at Inveruglas generating station, except that the courses are 10 in. and 14 in. alternately and that ordinary Portland cement is used throughout. The pointing is mortar to which a

to those at Clunie, except that the courses of precast slabs are each 22 in. deep. A shutter 8 ft. high was used on the inside face of the cast-in-situ reinforced concrete walls and the slabs erected as permanent shuttering on the outer face, each lift of concrete corresponding to the height of one course of slabs.

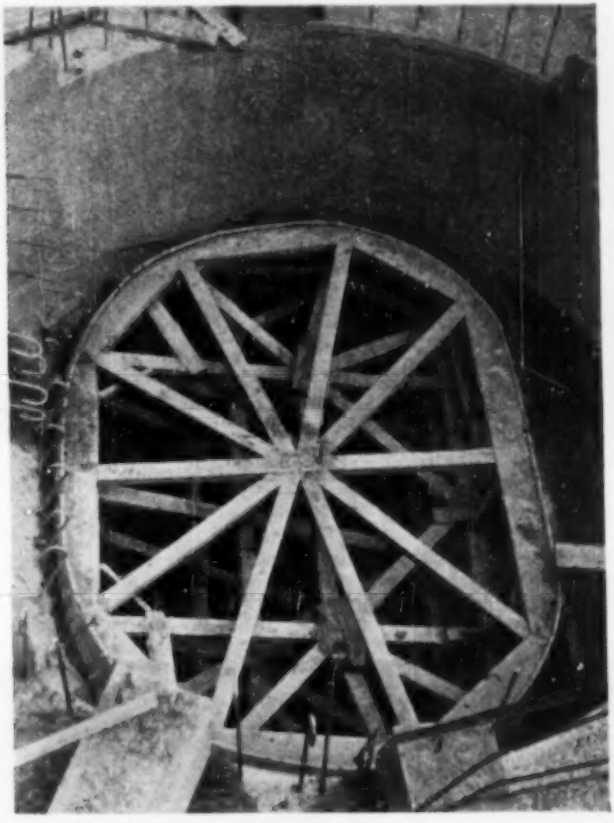


Fig. 9.—Clunie Generating Station: Shuttering for Draught Tubes.

burnt-sienna pigment is added. The coat of arms of the North of Scotland Hydro-electric Board above the entrance is also in concrete precast in sections in plaster of Paris piece-moulds.

The generating station at Pitlochry, with a capacity of 15,000 kw., is combined with the dam (*Fig.* 8). The walls of the generating station are of similar design

The dam at Pitlochry is about 480 ft. long and 50 ft. high and contains about 43,000 cu. yd. excluding the generating station. The cut-off wall is more than 1000 ft. long, and in parts is 160 ft. deep owing to the great depth to which the rock dips on the left bank. The deepest parts of the trench were lined with heavy steel-sheet piles 40 ft. long, which were with-

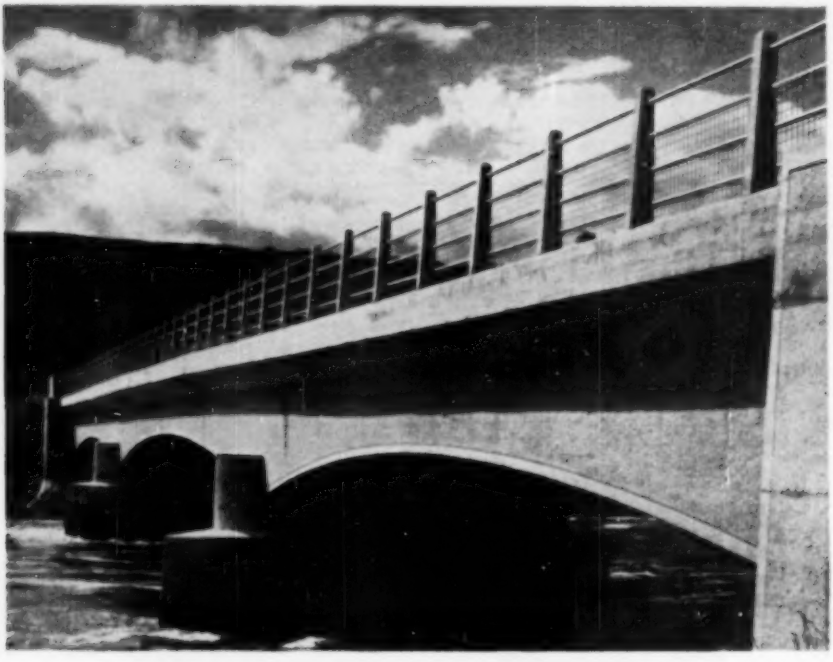


Fig. 10.—Aldour Bridge, near Pitlochry.

drawn as concreting proceeded. The walings were 9-in. by 3½-in. steel channels at 7-ft. centres vertically. The timber struts were 12 in. square, and were provided alternately at 10-ft. 6-in. and 7-ft. 9-in. centres horizontally. The top

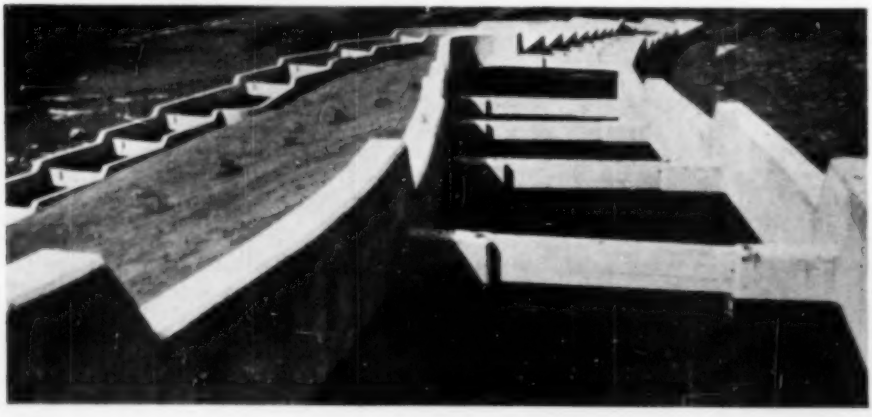


Fig. 11.—Fish-pass, Clunie.

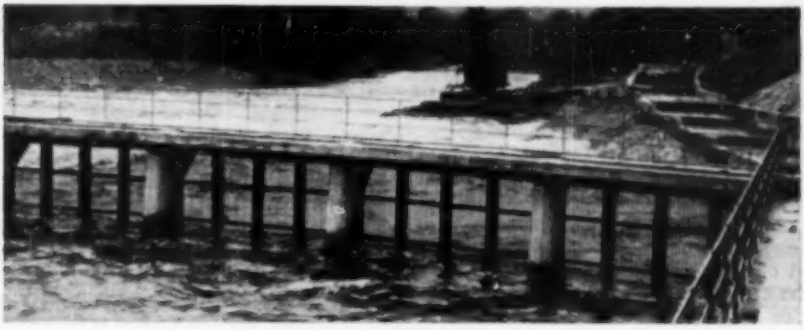


Fig. 12.—Tailrace Screen and Fish-pass, Pitlochry.

of the plain concrete cut-off wall, which has a minimum thickness of 5 ft., is surmounted by an access road.

Aldour Bridge.

The new Aldour bridge (*Fig.* 10), across the Tummel river ¾ mile below Pitlochry dam, is a reinforced concrete girder road-bridge with a total length of 300 ft. 6 in. The middle span of 94 ft. comprises two cantilevers and a suspended span 50 ft. long. Each end span is 77 ft. 6 in. long. Exposed concrete faces are bush-hammered. The ground at the site of the bridge is loose gravel and sand and the piers are carried on piles.

Fish-passes.

A feature of the dams at Clunie and Pitlochry are the fish-passes to enable salmon to by-pass the dams when proceeding upstream to spawn. The fish-passes are of the orifice type and comprise a series of compartments each at a slightly higher level than the preceding one, and connected by a submerged pipe or orifice in the lower part of the division wall through which the fish swim against the flow of the descending water. *Fig.* 11 shows the fish-pass at Clunie. The parts of the reinforced concrete walls in contact with water are painted with a black bituminous waterproof compound. The longitudinal walls and bottom were cast in situ. The cross walls have precast flash-boards fitting into slots in projections from the longitudinal walls. The fish-pass at Pitlochry is about 1000 ft. long and comprises 33 compartments most of which are 25 ft. long and 14 ft. wide. The depth of water is 8 ft. and the rise from one compartment to the next is 18 in. An observation chamber and counting device are incorporated in this fish-pass. The lower part of the pass is seen in *Fig.* 12, which also shows the tailrace screen (and supporting reinforced concrete structure) which prevents fish from entering the turbines.

The consulting engineers for the Loch Sloy works are Messrs. James Williamson & Partners and for the Tummel-Garry works Sir Alexander Gibb & Partners. The principal contractors for the structures are: Loch Sloy dam, Messrs. Balfour Beatty & Co., Ltd.; Inveruglas generating station, Messrs. Hugh Leggat, Ltd.; Clunie and Pitlochry dams and Aldour bridge, Messrs. George Wimpey & Co., Ltd.; Clunie tunnel, the Cementation Co., Ltd.; Clunie generating station, Messrs. A. M. Carmichael, Ltd. The precast concrete facing slabs for the generating stations at Inveruglas, Clunie, and Pitlochry were made by Messrs. George W. Bruce, Ltd.

Congress on Dams.

THE fourth congress of the International Commission on Large Dams is to be held at New Delhi, from January 10 to 15, 1951. The secretary of the British committee is Mr. C. H. Gray, 201 Grand Buildings, Trafalgar Square, London, W.C.2.

Book Reviews.

"The Economic Design of Rectangular Reinforced Concrete Sections." By T. P. O'Sullivan. (London: Sir Isaac Pitman & Sons, Ltd. 1950. Price 12s. 6d.)

THE author deals analytically with the problem of determining the least amount of reinforcement required in a reinforced concrete member of rectangular cross section subjected to a load acting parallel to the longitudinal axis and at a known point on either of the two principal axes of the cross section. The load may be a pull or a thrust, and may act within or outside the boundary of the section. The formulæ and charts are of general application within the limits described, and trial-and-error methods of determining the amount of reinforcement are eliminated. Some of the results of this purely mathematical treatment may be surprising to those accustomed to ordinary designs. For example, reinforcement disposed symmetrically may be extravagant if the point of application of the load is within the section. The derivation of the formulæ is given in an appendix. Excluding an appendix on frame analysis, which the author admits has no bearing on the main subject, the subject is dealt with comprehensively in fewer than 60 pages.

"Fundamentals of Reinforced Concrete." By K. F. Antia. (Bombay: The Concrete Association of India. 1950. Price 17/8 rupees (about £1 6s. 6d.).

THIS book of 670 pages, with a good index and numerous illustrations, aims at dealing with the elements of the whole subject of reinforced concrete within a single volume, and gives good descriptions of some of the latest methods. Part I (307 pages) deals with materials and construction. Part II (363 pages) deals with design. The book is clearly printed and interesting to read. The English and the cross-references are excellent. The diagrams are generally clear. The photographs are well chosen, and those in the introductory chapter and some others show the high standard of reinforced concrete in India. The book is based largely on English, American, and Indian codes, the quotations from which are of value for ready reference.

Part I gives excellent information on materials, on the proportioning of aggregates, on the properties of concrete, on mixing, placing, compacting, and curing, on the fabrication of reinforcement, and on shuttering. Useful suggestions on the possibilities of precast construction are included. In reference to a statement concerning submersible bridges, it may be mentioned that many such bridges have been built in India in masonry as well as in reinforced concrete. There is an apt reference to that ingredient, so necessary in the production of good concrete, contained in the experience and conscientiousness of those taking part in its production. The evils resulting from dropping concrete from a height and the effects of inadequate curing are well described; these subjects deserve greater consideration in this country. Defective construction joints are mentioned, but are not well shown in the illustrations. Blemishes due to rendering because of defective shuttering are described.

Part II deals with design. The fundamentals are given, with worked examples which would have been of greater value to the novice had they been accompanied by some good working details. In the chapter on arches it is stated, "In the case of semi-circular arches, there is no horizontal thrust," but no explanation or proof is given in substantiation.

Prestressed concrete is interestingly described in a chapter of 32 pages. The treatment of rigid frames by moment-distribution methods is very briefly dealt with, and the signs of the bending moments and consequent deformation of the frame require further explanation. The horizontal forces also require balancing. It is not clear why 3 in. cover has to be added on both faces of structures to contain materials, thereby necessitating a thickness of 18 in. for an effective depth of 12 in. "Shell" construction, piles, and culverts are alluded to, but reinforced concrete piles in particular receive small notice.

The book, which concludes with a chapter on repairs, contains errors in the mathematics which often make the work difficult to follow and sometimes spoil the worked examples, and a table referred to on page 604 is not given in the book.

R. P. M.

Equivalent Elastic Systems in the Analysis of Continuous Structures.—II.*

By L. A. BEAUFOY, Ph.D., M.Sc., A.M.Inst.C.E., M.I.Mech.E., M.I.Struct.E., and A. F. S. DIWAN, Ph.D., B.Sc.

THE method of using equivalent elastic systems will be illustrated by application to the continuous frame on flexible columns shown in *Fig.* 6. The distribution of bending moments for the given load on span CF will be computed and the influence line for the bending moment at E determined. Axial deformation of the members will be neglected.

Structural Analysis.

The first step is to evaluate the elastic constants of a rafter and a column, and the stiffness factors for the ends of these members. The values are given in column (1) of *Table* No. II. Start at one end of the frame and consider the

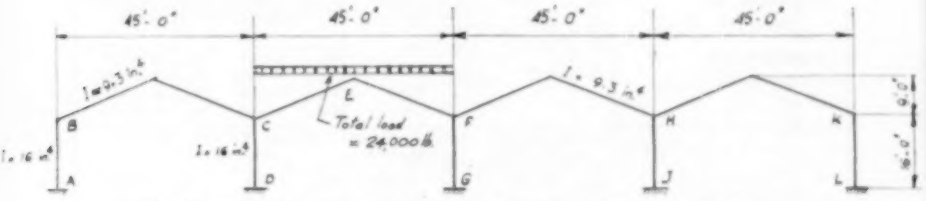


Fig. 6.—Continuous-gable Arch Frame on Flexible Columns.

partial equivalent system No. 1 in *Fig.* 7 comprising the real member HK and a virtual equivalent system No. 1 for member KL, which in this case is the member KL. Enter in column (2), *Table* No. II, under virtual equivalent system No. 1, the stiffness factors for the end K of member KL and the elastic constants for member KL. Then compute, as in the following, the elastic constants for the combination constituting the partial equivalent system No. 1. From equations (8)—see Part I—and referring the co-ordinates $\bar{x}$ and $\bar{y}$ to joint H:

$$S = 5{\cdot}2 + 1{\cdot}0 = 6{\cdot}20$$

$$\bar{x} = \frac{(5{\cdot}2 \times 22{\cdot}5) + (1{\cdot}0 \times 45)}{6{\cdot}2} = 26{\cdot}14\,; \quad \bar{y} = \frac{(5{\cdot}2 \times 4{\cdot}5) - (1{\cdot}0 \times 8)}{6{\cdot}2} = 2{\cdot}484$$

$$I_x = 35{\cdot}15 + 21{\cdot}3 + 5{\cdot}2(4{\cdot}5 - 2{\cdot}484)^2 + 1{\cdot}0(8 + 2{\cdot}484)^2 = 187{\cdot}6$$
$$I_y = 878 + 5{\cdot}2(26{\cdot}14 - 22{\cdot}5)^2 + 1{\cdot}0(45 - 26{\cdot}14)^2 = 1302$$
$$I_{xy} = 5{\cdot}2(4{\cdot}5 - 2{\cdot}484)(22{\cdot}5 - 26{\cdot}14) + 1{\cdot}0(-2{\cdot}484 - 8)(45 - 26{\cdot}14) = -236$$

$$I'_x = 187{\cdot}6 - \frac{236^2}{1302} = 144{\cdot}6\,; \quad I'_y = 1302 - \frac{236^2}{187{\cdot}6} = 1005$$

Enter these values in column (3), *Table* No. II. From equations (7), calculate the forces and moment required at joint H to produce unit rotation there.

$$m_\phi = \frac{1}{6{\cdot}2} + \frac{(2{\cdot}484)^2}{144{\cdot}6} + \frac{(26{\cdot}14)^2}{1005} - \frac{2 \times 2{\cdot}484 \times 26{\cdot}14(-236)}{144{\cdot}6 \times 1302} = 1{\cdot}047$$

* Concluded from November, 1950.

TABLE II.—ELASTIC CONSTANTS, STIFFNESS FACTORS, AND INDUCED MOVEMENTS.

1			2		3		4		5	
	RAFTER HK	COLUMN KL	Virtual Equivalent systems				Impose Unit rotation on left-hand end of the partial equivalent system		Impose Unit horiz. translation on left-hand end of the partial equivalent system	
			Virtual Equivalent system No 1		Partial Equivalent system No 1		Actual Stiffness Factors Member HK			
$\bar{s}$	5·2	1·0	m_ϕ	4·0	$\bar{s}$	6·20	m_ϕ	1·047	m_Δ	0·0500
$\bar{x}$	22·5 *	- ϕ	h_ϕ	0·375	$\bar{x}$	26·14	h_ϕ	0·0500	h_Δ	0·0069
$\bar{y}$	4·5 *	-8·0 ϕ	h_o	0·0469	$\bar{y}$	2·484	V_ϕ	-0·0292	V_Δ	-0·001253
I_x	35·15	21·33	$\bar{s}$	1·00	I_x	187·6	Induced Movements Joint K			
I_y	878·0	-	$\bar{y}$	-8·00	I_y	1302				
I_{xy}	-	-	I_x	21·33	I_{xy}	-236	m_o	0·134	m_o	0·0486
m'_ϕ	1·344	4·0			I'_x	144·6	h_o	0·0500	h_o	0·0069
h'_ϕ	0·128	-0·375			I'_y	1005	ϕ_ϕ	0·1349	ϕ_Δ	0·0487
h'_Δ	0·0284	0·0469					Δ_ϕ	2·1390	Δ_Δ	0·336
			Virtual Equivalent system No 2		Partial Equivalent system No 2		Actual Stiffness Factors Member FH			
NOTES			m_ϕ	5·047	$\bar{s}$	5·527	m_ϕ	1·098	m_Δ	0·0663
* ORIGIN AT JOINT H			h_ϕ	-0·325	$\bar{x}$	23·8	h_ϕ	0·0663	h_Δ	0·01192
ϕ ORIGIN AT JOINT K			h_Δ	0·0538	$\bar{y}$	3·86	V_ϕ	-0·0278	V_Δ	-0·000856
			$\bar{s}$	0·327	I_x	89·9	Induced Movements Joint H			
			$\bar{y}$	- 6·04	I_y	1034				
			I_x	18·6	I_{xy}	- 74·3	m_o	0·261	m_o	0·1022
					I'_x	85·9	h_o	0·0663	h_o	0·01192
					I'_y	972	ϕ_ϕ	0·0842	ϕ_Δ	0·0331
							Δ_ϕ	1·758	Δ_Δ	0·424
			Virtual Equivalent system No 3		Partial Equivalent system No 3		Actual Stiffness Factors Member CF			
			m_ϕ	5·098	$\bar{s}$	5·488	m_ϕ	1·112	m_Δ	0·0721
			h_ϕ	-0·309	$\bar{x}$	23·65	h_ϕ	0·0720	h_Δ	0·0134
			h_Δ	0·0588	$\bar{y}$	3·99	V_ϕ	-0·02764	V_Δ	-0·000788
					I_x	78·1				
			$\bar{s}$	0·2876	I_y	1016·	Induced Movements Joint F			
			$\bar{y}$	-5·25	I_{xy}	- 59·8				
			I_x	17·01	I'_x	74·6	m_o	0·253	m_o	0·1072
					I'_y	970·	h_o	0·0720	h_o	0·0134
							ϕ_ϕ	0·0748	ϕ_Δ	0·0306
							Δ_ϕ	1·618	Δ_Δ	0·387
			Virtual Equivalent system No 4		Partial Equivalent system No 4		Actual Stiffness Factors Member BC			
			m_ϕ	5·112	$\bar{s}$	5·479	m_ϕ	1·129	m_Δ	0·0736
			h_ϕ	-0·003	$\bar{x}$	23·65	h_ϕ	0·0736	h_Δ	0·01378
			h_Δ	0·0603	$\bar{y}$	4·018	V_ϕ	- 0·0275	V_Δ	0·000772
					I_x	75·8				
			$\bar{s}$	0·279	I_y	1012·	Induced Movements Joint C			
			$\bar{y}$	- 5·03	I_{xy}	- 56·8				
			I_x	16·60	I'_x	72·6	m_o	0·261	m_o	0·1083
					I'_y	969·	h_o	0·0736	h_o	0·01378
							ϕ_ϕ	0·0724	ϕ_Δ	0·0300
							Δ_ϕ	1·586	Δ_Δ	0·379

$$h_\phi = \frac{2\cdot484}{144\cdot6} - \frac{26\cdot14(-236)}{144\cdot6 \times 1302} = 0\cdot050\,; \quad v_\phi = \frac{2\cdot484(-236)}{1005 \times 187\cdot6} - \frac{26\cdot14}{1005} = -0\cdot0292$$

Enter these values of the rotational stiffness factors in column (4). Transfer these forces to the elastic centre O_1 of virtual equivalent system No. 1 as follows:

$$m_o = 1\cdot047 + (0\cdot05 \times 8) - (0\cdot292 \times 45) = 0\cdot134\,; \quad h_o = h_\phi = 0\cdot050.$$

Substitute in equations (1) to give

$\phi_\phi = 0\cdot134 \times 1\cdot0 = 0\cdot134$ and $\Delta_\phi = (0\cdot050 \times 21\cdot3) + (8 \times 0\cdot134) = 2\cdot139$, which are the induced movements at joint K resulting from unit rotation imposed at joint H, and are also entered in column (4).

Similarly, calculate the horizontal displacement stiffness factors and the induced movements at joint K for unit horizontal displacement of joint H, and enter the values in column (5), *Table* No. II.

Next consider partial equivalent system No. 2 (*Fig.* 7), which comprises the real member FH and a virtual equivalent system No. 2 for members HJ, HK, and KL. Combine partial equivalent system No. 1, which is the sum of members HK and KL in series, with member HJ in parallel at joint H to obtain the properties of virtual equivalent system No. 2. This is done by adding together the relevant stiffness factors of partial equivalent system No. 1 and of member HJ to obtain the stiffness factors at the common joint H of the combination, which constitutes partial equivalent system No. 2. Therefore

$M_\phi = 4{\cdot}0 + 1{\cdot}047 = 5{\cdot}047$;
$H_\phi = 0{\cdot}05 - 0{\cdot}375 = -0{\cdot}325$; and $H_\delta = 0{\cdot}0069 + 0{\cdot}0469 = 0{\cdot}0538$.

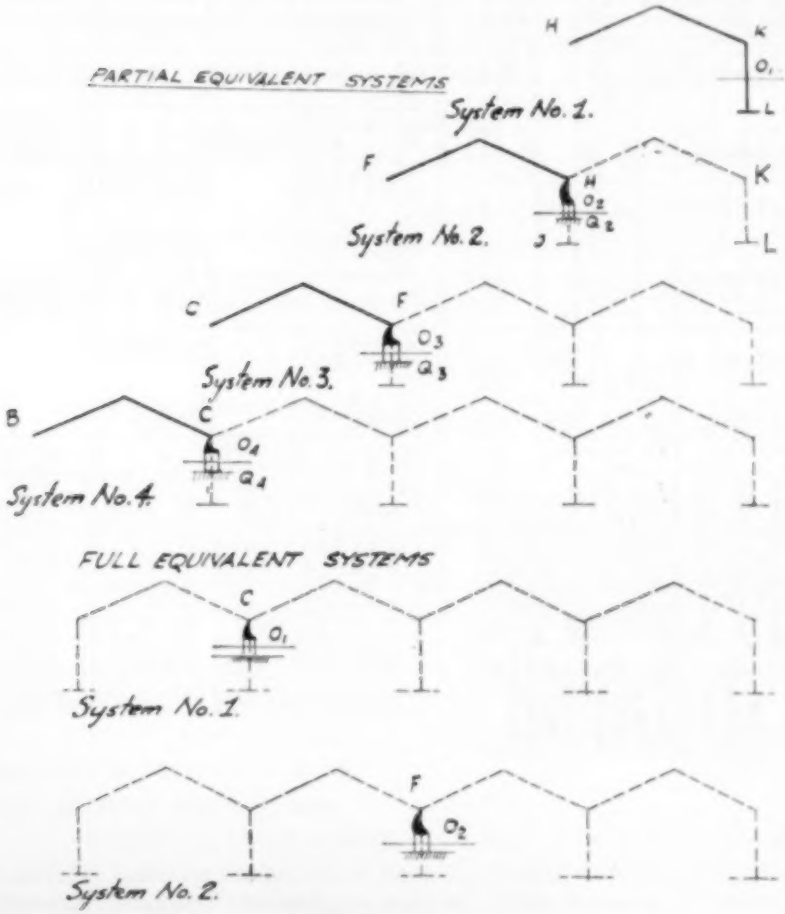


Fig. 7.

TABLE III.—DISPLACEMENTS.

		Joint B		Joint C		Joint F		Joint H		Joint K	
		φ	Δ	φ	Δ	φ	Δ	φ	Δ	φ	Δ
UNIT DISPLACEMENTS	Imposed unit displacement at joint H	0·0479 0·0127	0·570 0·1496	0·0598 0·0154	0·818 0·218	0·0748 0·0306	1·618 0·387	1·00 –	– 1·00	0·1569 0·0487	2·139 0·536
	Imposed unit displacement at joint F	0·0970 0·0251	1·122 0·298	0·0842 0·0331	1·738 0·424	1·00 –	– 1·00	0·0842 0·0331	1·738 0·424	0·0970 0·0251	1·122 0·298
	Imposed unit displacement at joint B	1·00 –	– 1·00	0·0724 0·0300	1·586 0·379	0·0539 0·0158	0·731 0·1935	0·0287 0·0076	0·404 0·1072	0·0236 0·0063	0·276 0·0757
TOTAL JOINT DISPLACEMENTS FOR LOAD ON CEF	Balancing Movements at joint C and resulting induced movements elsewhere	-0·87 -12·07	-13·8 -132·7	-6·44 –	– -247·4	-0·48 -7·58	-10·4 -35·8	-0·38 -3·81	-5·27 -56·0	-0·31 -3·14	-3·7 -37·0
	Balancing Movements at joint F and induced movements elsewhere	0·51 5·72	5·9 68·0	0·44 7·55	9·2 96·8	5·22 –	– 228·1	0·44 7·55	9·2 96·9	0·51 5·72	5·9 68·0
	Total joint displacement	-6·71	-72·6	1·55	-141·4	-2·84	121·9	3·80	46·5	2·78	33·2
DISPLACEMENTS FOR UNIT RELATIVE ROTATION AT E	Balancing Movements at C and induced movements at B	0·0105 0·0631	0·167 0·696	0·0779 –	– 1·295						
	Balancing Movements at F and induced movements at H and K					-0·0616 –	– 1·063	-0·0052 -0·0352	-0·108 -0·450	-0·0060 -0·0267	-0·069 -0·317
	Total joint displacement	0·0736	0·861	0·0779	1·295	-0·0616	-1·063	-0·0404	-0·558	-0·0327	-0·386

TABLE IV.—CALCULATION OF BENDING MOMENTS AND HORIZONTAL FORCES.

		BA	BC	CB	CD	CF	FC	FG	FH	HF	HJ	HK	KH	KL
BENDING MOMENTS	FIXED-END MOMENTS	0	0	0	0	-22·5	22·5	0	0	0	0	0	0	0
	JOINT ROTATION (m'_ϕ × total φ; carry-over C = 0·48)	-28·8	-9·0 -0·30	2·1 1·3	6·2	2·1 0·50	-3·8 -0·3	-11·4	-3·8 -0·7	5·1 0·5	15·2	5·1 -0·5	3·7 -0·7	11·0
	JOINT TRANSLATION (m'_Δ × total Δ; carry-over C = 1·0	27·2	-9·2 18·0	-18·0 9·2	53·0	-18·0 -15·5	15·5 18·0	-45·7	15·5 -8·0	6·0 -15·5	-17·4	6·0 -4·3	4·3 -6·0	-12·5
	TOTAL MOMENTS	0·4	-0·5	-5·4	59·2	-53·4	51·9	-57·1	5·0	-3·9	-2·2	6·3	1·3	-1·4
		AB			DC			GF			JH			LK
	carry-over (top joint rotation)	-13·4			3·1			-5·7			7·6			5·6
	carry-over (top joint translation)	27·2			53·0			-45·7			-17·4			-12·5
	TOTAL MOMENTS	13·8			56·1			-51·4			-9·8			-6·9
HORIZONTAL FORCES		BA	BC	CB	CD	CF	FC	FG	FH	HF	HJ	HK	KH	KL
	FIXED-END THRUST	0	0	0	0	15·0	-15·0	0	0	0	0	0	0	0
	JOINT ROTATION (h'_ϕ × total φ; carry-over C = 1·0	2·52	-0·86 -0·20	0·20 0·86	-0·58	0·20 0·36	-0·36 -0·20	1·06	-0·36 -0·49	0·49 0·36	-1·43	0·49 -0·36	0·36 -0·49	-1·04
	JOINT TRANSLATION (h'_Δ × total Δ; carry-over C = 1·0	-3·41	-2·07 4·02	-4·02 2·07	6·62	-4·02 -3·47	3·47 4·02	5·71	3·47 -1·32	1·32 -3·47	2·18	1·32 -0·34	0·94 -1·32	1·36
	TOTAL HORIZONTAL FORCE	-0·89	0·89	-0·89	7·20	8·07	-8·07	6·77	1·50	-1·50	0·75	0·51	-0·51	0·52
		AB			DC			GF			JH			LK
	carry-over (top joint rotation)	-2·52			0·58			-1·06			1·43			1·04
	carry-over (top joint translation)	3·41			6·62			-5·71			-2·18			-1·36
	TOTAL HORIZONTAL FORCE	0·89			7·20			-6·77			-0·75			-0·32

NOTE.—In *Tables* III and IV, for bending moments 1 unit = 1000 ft.-lb., and for forces 1 unit = 1000 lb.

Substitute in the equations in column (4) of *Table* No. I (which was given in Part I) to obtain the elastic constants of partial equivalent system No. 2:

$$S = \frac{0{\cdot}0538}{(5{\cdot}047 \times 0{\cdot}0538) - (0{\cdot}325)^2} = 0{\cdot}327;$$

$$\bar{y} = \frac{-0{\cdot}325}{0{\cdot}0538} = -6{\cdot}04; \text{ and } I_x = \frac{1}{0{\cdot}0538} = 18{\cdot}6.$$

Enter these values in column (2), *Table* No. II.

A complete cycle of operations has now been described. The process is continued to complete *Table* No. II. From the induced movements in this table, the unit displacements in the upper part of *Table* No. III are readily obtained and show the rotations and translations induced at each joint by unit rotation or translation imposed at any joint. *Fig.* 7 illustrates the four partial equivalent systems used in the foregoing procedure. In each case the virtual equivalent system shown diagrammatically replaces the members indicated by the broken lines. The structural analysis is now complete and it may be used to determine the distribution of moments and forces due to load or to plot any influence line.

Stress Analysis.

The stiffness-factors for the joints of the frame can be obtained from *Table* No. II.

For joint C: $M_\phi = 6{\cdot}159$; $H_\phi = -0{\cdot}253$; $H_\Delta = 0{\cdot}0672$.
For joint F: $M_\phi = 6{\cdot}196$; $H_\phi = -0{\cdot}2424$; $H_\Delta = 0{\cdot}07074$.

Hence for full equivalent system No. 1 (*Fig.* 7), using the expressions in column (4) of *Table* No. I, the elastic constants are

$$S = \frac{0{\cdot}0672}{(6{\cdot}159 \times 0{\cdot}0672) - (0{\cdot}253)^2} = 0{\cdot}190;$$

$$\bar{y} = \frac{-0{\cdot}253}{0{\cdot}0672} = -3{\cdot}76; \text{ and } I_x = \frac{1}{0{\cdot}0672} = 14{\cdot}89.$$

Similarly for full equivalent system No. 2 (*Fig.* 7), $S = 0{\cdot}1869$; $\bar{y} = -3{\cdot}435$; $I_x = 14{\cdot}14$. Note that in each case the virtual equivalent systems shown diagrammatically in *Fig.* 7 replace the members indicated by the broken lines, that is all the members of the frame.

For the given load on the frame, if all the joints are considered fixed, the fixed-end moments are − 22,500 ft.-lb. at CF, and + 22,500 ft.-lb. at FC. The fixed-end thrusts are + 15,000 lb. at CF, and − 15,000 lb. at FC. Transfer these unbalanced components at the joints C and F to the elastic centres of full equivalent systems Nos. 1 and 2 respectively. For joint C,

$m_0 = +22{,}500 + 15{,}000 \times (-3{\cdot}76) = -33{,}900$ ft.-lb., and $h_0 = -15{,}000$ lb.

Substitution in equations (1) gives

$\phi = (-33{,}900) \times 0{\cdot}19 = -6440$;

$$\Delta = (-15{,}000 \times 14{\cdot}89) - [(-6440) \times (-3{\cdot}765)] = -247{,}400$$

which are the balancing movements at joint C. Similarly, for joint F the balancing movements are $\phi = 5220$ and $\Delta = 228{,}100$. These balancing movements are entered in the middle part of *Table* No. III, but in units of 1000 lb. in which

form the figures are more easily manipulated. The movements induced elsewhere are determined by proportion from the unit-displacement table, and hence the total joint displacements are found.

Bending Moments and Horizontal Forces.

The calculation of bending moments and horizontal forces follows readily as shown in *Table* No. IV. A check on the arithmetical work is that the sum of the moments at each joint and the sum of the horizontal reactions should both be equal to zero. It is seen that the agreement is good.

Influence Line for Bending Moment at E.

The structural analysis in the foregoing applies if it is required to plot the influence line for the bending moment at joint E. For the stress analysis the whole frame is replaced by a full equivalent system composed of the real member CF, and virtual equivalent systems CQ_2 and FQ_3, all in series (*Fig.* 8*a*), which are the same as those shown as partial equivalent systems Nos. 2 and 3 respectively in *Fig.* 7 the elastic constants for which are given in *Table* No. II. The elastic

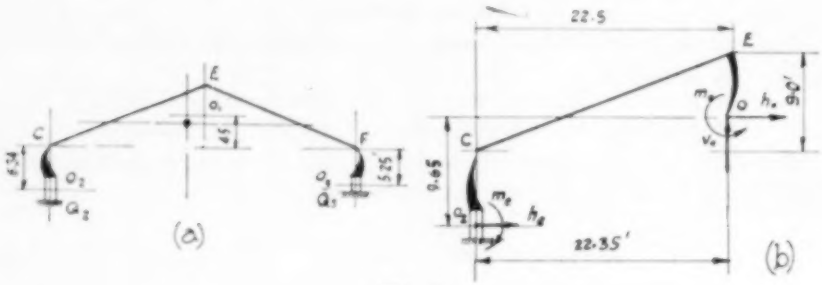


Fig. 8.

constants for CF are as given for HK in column (1), *Table* No. II. If the origin for $\bar{x}$ and $\bar{y}$ is C for the virtual equivalent system CQ_2 and the real member CF, and is F for the virtual equivalent system FQ_3, then for CQ_2, $\bar{S} = 0{\cdot}327$, $\bar{y} = -6{\cdot}04$, and $I_x = 18{\cdot}6$; for FQ_3, $\bar{S} = 0{\cdot}2875$, $\bar{y} = -5{\cdot}25$, and $I_x = 17{\cdot}01$. From equations (8) the elastic constants of the resultant system are as follows if the origin of $\hat{x}$ and $\hat{y}$ is at C:

$$\hat{S} = 0{\cdot}327 + 0{\cdot}2876 + 5{\cdot}2 = 5{\cdot}8146$$

$$\hat{x} = \frac{(5{\cdot}2 \times 22{\cdot}5) + (0{\cdot}2876 \times 45)}{5{\cdot}815} = 22{\cdot}35$$

$$\hat{y} = \frac{0{\cdot}327(-6{\cdot}24) + 0{\cdot}2876(-5{\cdot}25) + (5{\cdot}2 \times 4{\cdot}5)}{5{\cdot}815} = 3{\cdot}41$$

$$I_x = 18{\cdot}6 + 17{\cdot}01 + 35{\cdot}15 + 0{\cdot}327(9{\cdot}65)^2 + 0{\cdot}2876(8{\cdot}66)^2 + 5{\cdot}2(1{\cdot}09)^2 = 128{\cdot}97$$

$$I_y = 878 + 0{\cdot}327(22{\cdot}35)^2 + 5{\cdot}2(0{\cdot}15)^2 + 0{\cdot}2876(22 - 65)^2 = 1188{\cdot}4.$$

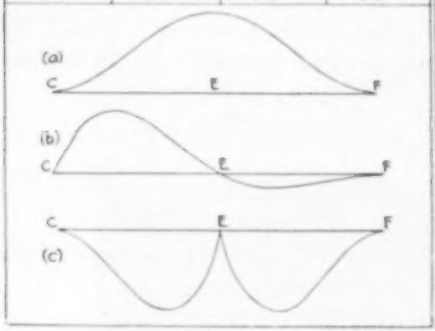
$$I_{xy} = 0{\cdot}327(-22{\cdot}35)(-9{\cdot}65) + (5{\cdot}2 \times 0{\cdot}15 \times 1{\cdot}09) + [0{\cdot}2876 \times 22{\cdot}65(-8{\cdot}66)] = 14{\cdot}95$$

$$I'_x = 128{\cdot}97 - \frac{(14{\cdot}95)^2}{1188} = 128{\cdot}7\,; \quad I'_y = 1188 - \frac{(14{\cdot}95)^2}{128{\cdot}97} = 1187{\cdot}0.$$

TABLE V.

Horizontal distance from left-hand end of rafter (ft)	Deflection for unit horizontal displacement of left-hand end (a)	Deflection for unit rotation of left-hand end (b)	Deflection for unit relative rotation at E (c)
0·0	0·0	0·0	0·0
4·5	0·13	-5·06	-0·56
9·0	0·44	-5·78	-1·08
13·5	0·81	-2·97	-1·62
18·0	1·12	-1·48	-1·44
22·5	1·25	0·0	0·0
27·0	1·12	0·72	-1·44
31·5	0·81	0·81	-1·62
36·0	0·44	0·54	-1·08
40·5	0·13	0·18	-0·56
45·0	0·0	0·0	0·0

Horizontal distance from C (ft)	Deflection for horizontal translation	Deflection for rotation of joint C	Deflection for rotation of joint F	Deflection for unit relative rotation at E	Total Deflection
0·0	0·0	0·0	0·0	0·0	0·0
4·5	0·305	-0·256	0·011	-0·56	-0·278
9·0	1·032	-0·290	0·055	-1·08	-0·505
13·5	1·900	-0·188	0·050	-1·62	-0·108
18·0	2·626	-0·111	0·044	-1·44	1·116
22·5	2·94	0·0	0·0	0·0	2·94
27·0	2·626	0·055	-0·089	-1·44	1·152
31·5	1·900	0·062	-0·182	-1·62	0·160
36·0	1·032	0·041	-0·252	-1·08	-0·259
40·5	0·305	0·014	-0·188	-0·56	-0·228
45·0	0·0	0·0	0·0	0·0	0·0



The co-ordinates of E with reference to the elastic centre O of the combined system are $x_E = +0{\cdot}15$ ft. and $y_E = +5{\cdot}59$ ft., so that the centroidal moment and forces required to impose a unit relative rotation at E are

$$m_0 = \frac{I}{S} = \frac{I}{5{\cdot}815} = 0{\cdot}0172$$

$$h_0 = \frac{-y'_E}{I'_x} = \frac{-y_E + x_E\dfrac{I_{xy}}{I_y}}{I'_x} = \frac{-5{\cdot}59 + 0{\cdot}15\dfrac{14{\cdot}95}{1188}}{128{\cdot}7} = -0{\cdot}0435$$

$$v_0 = \frac{x_E - y_E\dfrac{I_{xy}}{I_x}}{I'_y} = \frac{0{\cdot}15 - 5{\cdot}59\dfrac{14{\cdot}94}{128{\cdot}9}}{1187} = -0{\cdot}000416$$

According to the sign convention, the directions of these centroidal quantities are as shown in *Fig. 8b*. Transfer these forces to the elastic centres O_2 and O_3 in turn. For point C, $m = -0{\cdot}172 + (0{\cdot}0435 \times 9{\cdot}65) - (0{\cdot}000416 \times 22{\cdot}35) = +0{\cdot}2382$; $h = +0{\cdot}0435$. From equation (1), the displacements at joint C due to these forces at O_2 are

$\phi = 0{\cdot}2382 \times 0{\cdot}327 = 0{\cdot}0779$; $\Delta = (0{\cdot}0779 \times 6{\cdot}24) + (0{\cdot}0435 \times 18{\cdot}6) = 1{\cdot}295$.

The displacements at joint C induce movements at joints to the left of C only. Calculate these movements by proportion from the data in the upper part of *Table* No. III and enter them in the lower part of *Table* No. III. Similarly, calculate the displacements at joint F and the movements induced at joints to

the right of F. Hence obtain the total joint displacements (in the lower part of *Table* No. III) due to the imposed unit relative rotation at E.

Finally, compute the influence line for M_E as in *Table* No. V from the deflection diagrams for a single rafter for the cases given on the left-hand side of *Table* No. V, namely, (*a*) Unit horizontal translation of one end of the member, the other end being fixed; (*b*) Unit rotation of one end of the member, the other end being fixed; (*c*) Unit relative rotation at the section of the member (in this case E) for which the influence line is required, both ends being fixed. The ordinates of the deflection diagrams are given in *Table* No. V. The influence line is computed from these ordinates by proportion as on the right-hand side of *Table* No. V, the actual displacements of the terminals of each rafter being known.

The value of M_E for the given load is then obtained from the influence line as the area under the influence line between C and F multiplied by the load per foot of span. Therefore,

$$M_E = \frac{24{,}000}{45}\{[\text{Area under } (a) \times (1{\cdot}295 + 1{\cdot}063)] + [\text{area under } (b) \times (0{\cdot}0779 + 0{\cdot}0616)] + [\text{area under } (c)]\}$$

$$= (15{,}000 \times 2{\cdot}358) - (22{,}500 \times 0{\cdot}1395) - 22{,}500 = 9730 \text{ ft.-lb.}$$

This bending moment should be compared with that derived independently from *Table* No. IV, namely,

$$M_E = [-53{\cdot}4 + 0{\cdot}75 - (9 \times 8{\cdot}07) + 135]1000 = 9{,}720 \text{ ft.-lb.}$$

"Concrete and Constructional Engineering" Prize Design.

THE "CONCRETE AND CONSTRUCTIONAL ENGINEERING" prize of £25, which is awarded each year for competition amongst the students attending Professor A. L. L. Baker's course of concrete technology at the Imperial College of Science and Technology, City and Guilds College, has been awarded for the year 1948–1949 to Mr. K. A. Everard. The assessor was Mr. F. E. Wentworth-Sheilds, Past President of the Institution of Civil Engineers, who in the course of his report writes:

The subject of the competition was the development of a site on the Thames estuary, including a double-berth jetty for vessels of 15,000 tons, a bridge over the river together with approaches to the bridge, a ten-story warehouse, light factories, workers' flats, administration block, oil-storage tanks, and other structures needed for rail, road, and water access. The competitors worked in groups. Each group prepared a general plan of its scheme, and each member of a group made a report and drawings describing in detail one or more of the structures included in the scheme.

All the competitors showed a good grasp of the essentials of engineering design, and the reports and drawings gave evidence of careful and original work. Indeed, it has not been easy to award the prize, but I have come to the conclusion that the most deserving competitor is Mr. K. A. Everard who submitted designs for a two-berth jetty with road and rail approaches, a wharf, and a boiler house. I consider that honourable mention should be made of Mr. J. Lonergan for his design for a block of flats, of Mr. A. Gelibter for his design for a warehouse, and of Mr. S. Revesz for his design for an administration block.

The competition has again resulted in some very praiseworthy designs.

Reconstruction of a Quay at Wallsend.

THE quay at the Turbinia Works of Parsons Marine Steam Turbine Co., Ltd., on the north bank of the river at Wallsend-on-Tyne, has been reconstructed recently. The structure (*Fig.* 1), which is about 642 ft. long, comprises six parts (*Fig.* 3), namely, the new eastern extension 220 ft. 9 in. long, an old timber quay 114 ft. long, an old reinforced concrete quay 51 ft. 7 in. long, a new western extension 158 ft. 3 in. long, a launch-dock about 80 ft. long, and a new bollard-quay the greatest width of which is 17 ft. 9 in.

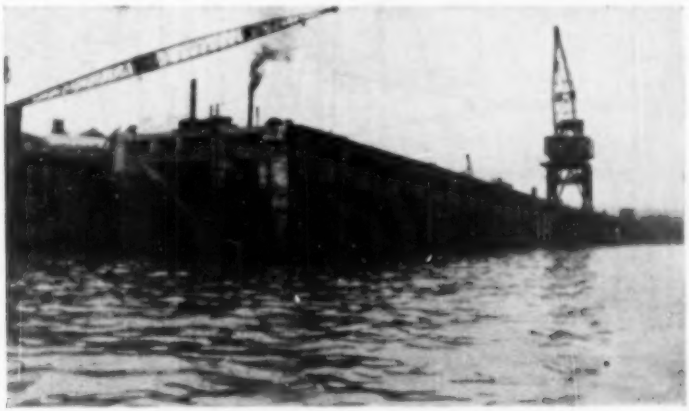


Fig. 1.—New Quay at Wallsend-on-Tyne.

Eastern and Western Extensions.

The extensions (*Figs.* 2 to 4) are similar in design and comprise two rows of vertical reinforced concrete piles at 18 ft. centres transversely, with an additional row of piles in the western extension to support the front of the deck which projects forward to the curved river-line stipulated by the Tyne Improvement Commissioners. Construction below high water is generally of precast reinforced concrete, and cast-in-situ reinforced concrete is used elsewhere. At a level sufficiently high above low water to enable construction to proceed at most states of low tide there are horizontal reinforced concrete braces transversely and longitudinally, and diagonal braces extend from this level to the head of the rear row of reinforced concrete piles. The ground and filling behind the wharf are retained by a wall of steel sheet-piles tied at the head by twin steel tie-rods connecting to a steel **I**-beam and a continuous reinforced concrete anchor slab 7 ft. 6 in. deep and 3 ft. thick (*Fig.* 5). The outward deflection of the sheet-piles imposes an outward thrust on the transverse horizontal braces; this is resisted by tension in the diagonal tie, which in turn transfers the force to the rear longitudinal beam of the deck in which ends of the tie-rods are anchored.

The deck comprises large rear and front longitudinal main beams which carry the crane rails, two smaller longitudinal beams for standard-gauge rail-track, and transverse beams. The structure is designed for a superimposed load of 672 lb. per square foot, a 30-tons crane on two four-wheeled bogies imposing a maximum load of 45 tons from each wheel of any one bogey, a 90-tons turbine on an eight-wheel truck drawn by a 48-tons locomotive, and berthing or mooring loads.

The reinforced concrete piles are 16 in. square by about 60 ft. long and were driven generally to a set of ten blows for a penetration of 1½ in., but the severity of the driving varied from 2 in. for ten blows to refusal. The piles bear on strata of gravels and sands above which are silts and sands; soft ground occurs in places below the gravel. The greatest working load on one pile is 54 tons. The piles were driven

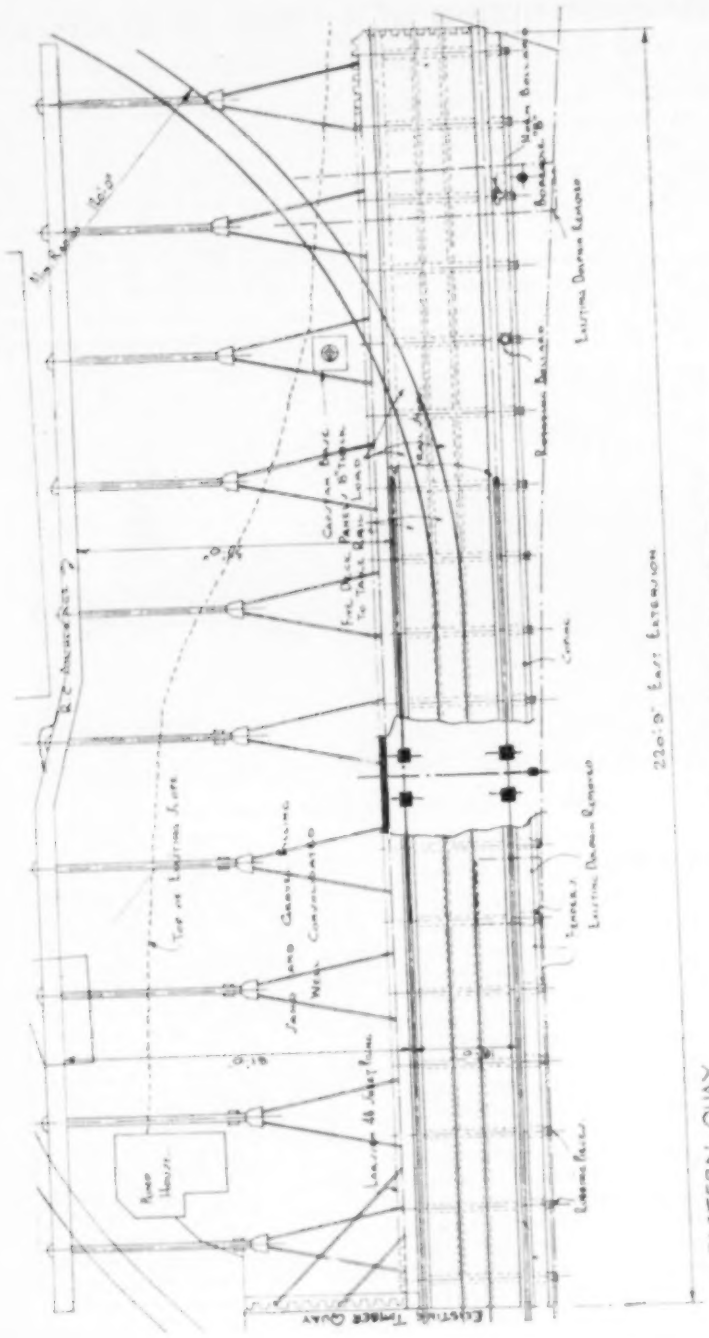


Fig. 2.—Plan of Eastern Quay.

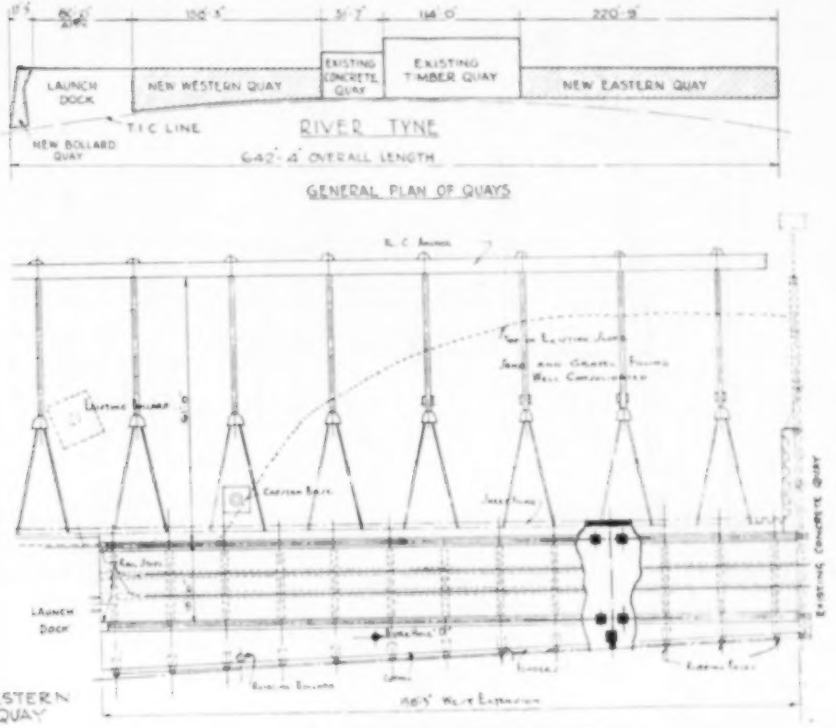


Fig. 3.—Plan of Quays and of Western Quay.

by a 3½-tons semi-automatic steam-hammer dropping 3 ft. 6 in. The longitudinal main beams were designed to distribute the loads over several piles.

The fendering comprises 12-in. square timbers horizontally and vertically with 12-in. by 4-in. vertical rubbing-pieces and a 12-in. by 6-in. oak coping. A precast reinforced concrete vertical fender-beam is suspended from the front main beam and built into the lower horizontal reinforced concrete longitudinal brace. Guards are provided at the bottom of the fenders to prevent ropes being caught behind the fenders.

The mixture of the concrete is 1 : 2 : 4 in work above water and 1 : 1½ : 3 in piles and other precast members below water. The design stresses are generally 18,000 lb. per square inch in mild steel reinforcement and 750 lb. per square inch in concrete, the modular ratio being 15. The coarse aggregate for the concrete was crushed whinstone. Natural sand and rapid-hardening Portland cement were used. The specified minimum strength of 6-in. cubes was 4500 lb. per square inch at fourteen days. This strength was easily exceeded. Some cubes tested at seven days could not be broken as the testing machine available could only cause a compressive stress of 6000 lb. per square inch. The whinstone often contained a considerable amount of dust. Batching was by volume.

The Bollard Quay.

The bollard-quay (*Fig.* 6) is a small separate structure at the extreme western (upstream) end of the quay, and is designed for a mooring pull in any direction. The structure is supported on six vertical piles and five piles driven to a rake of 1 in 3. The vertical upward component of the force on the bollard is resisted by the weight of the structure and the frictional resistance of the reinforced concrete piles and steel sheet-piles.

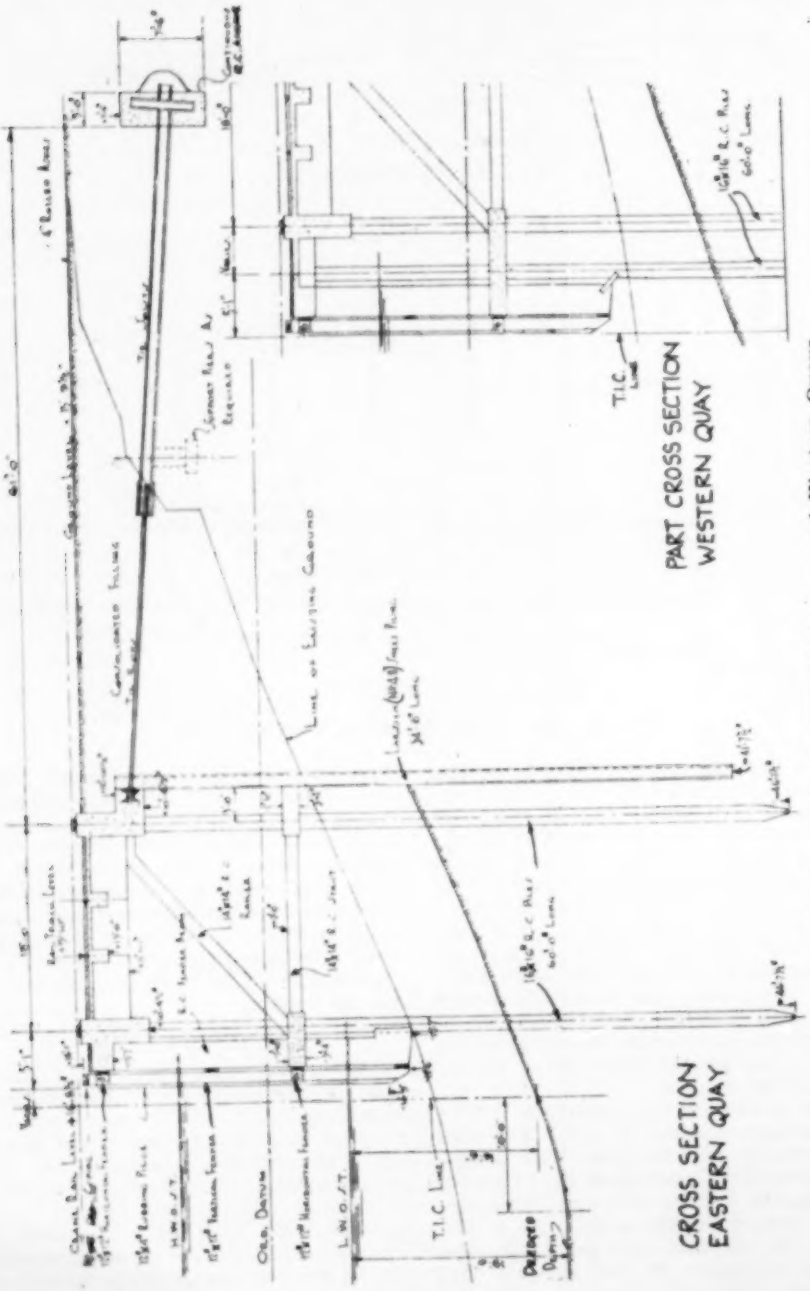


Fig. 4.—Cross Sections of Eastern and Western Quays.

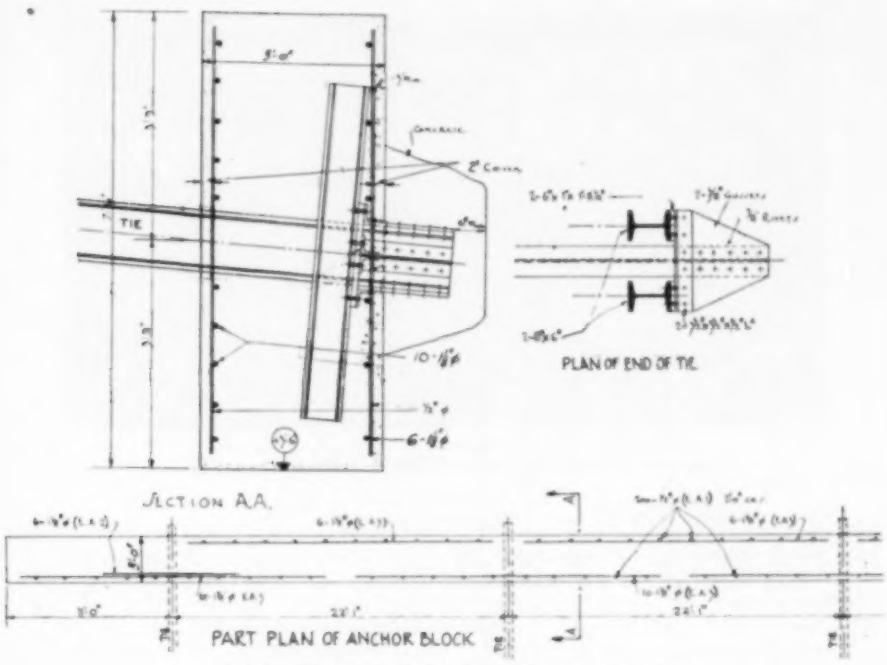


Fig. 5.—Reinforced Concrete Anchor Block.

Construction.

Before construction commenced two old dolphins and a gangway were demolished. The construction of the launch-dock entailed the excavation by grab of 1800 cu. yd. of silt 15 ft. below low water. The method of construction intended was altered because steel sheet-piles were not available at the start of the work, and therefore the reinforced concrete piles had to be driven at a maximum distance of 70 ft. from the top of the existing river bank before the sheet-piles were driven. The following procedure was adopted at the eastern end, and also with modifications at the western end. A timber piled gantry was constructed between the river bank and the new quay. A rail track was laid on the gantry and another track on the river bank, both tracks being parallel to the line of the new quay. The tracks carried a 10-tons travelling derrick crane, and beside this was a modified Bailey bridge set at right-angles to the tracks and cantilevering 40 ft. over the river. Rails laid along the top chords of the Bailey trusses carried a 50-ft. piling-frame and boiler. Thus the piling-frame could be taken over the river and the entire Bailey undercarriage could be moved laterally along the site. It was thought that when driving the outer row of piles it would be necessary to pack the undercarriage from the inner row of piles already driven in order to reduce whip and sway, but this was found to be unnecessary, although the piling-frame was on a cantilever a distance of 34 ft. from the outer rail. The piles were therefore driven with a long unsupported length from the bottom of the leaders to hard ground. *Fig.* 7 shows the arrangement of the piling-frame and the Bailey undercarriage used at the western end.

Some difficulty was experienced in casting the lower brace at the front of the quay. The bottom of this beam is about 2 ft. 6 in. above extreme low water, and was therefore too close to the water for sufficient frictional support to be provided for the shuttering clamped to the piles, especially as the piles were covered with black oil from the water. The shut-

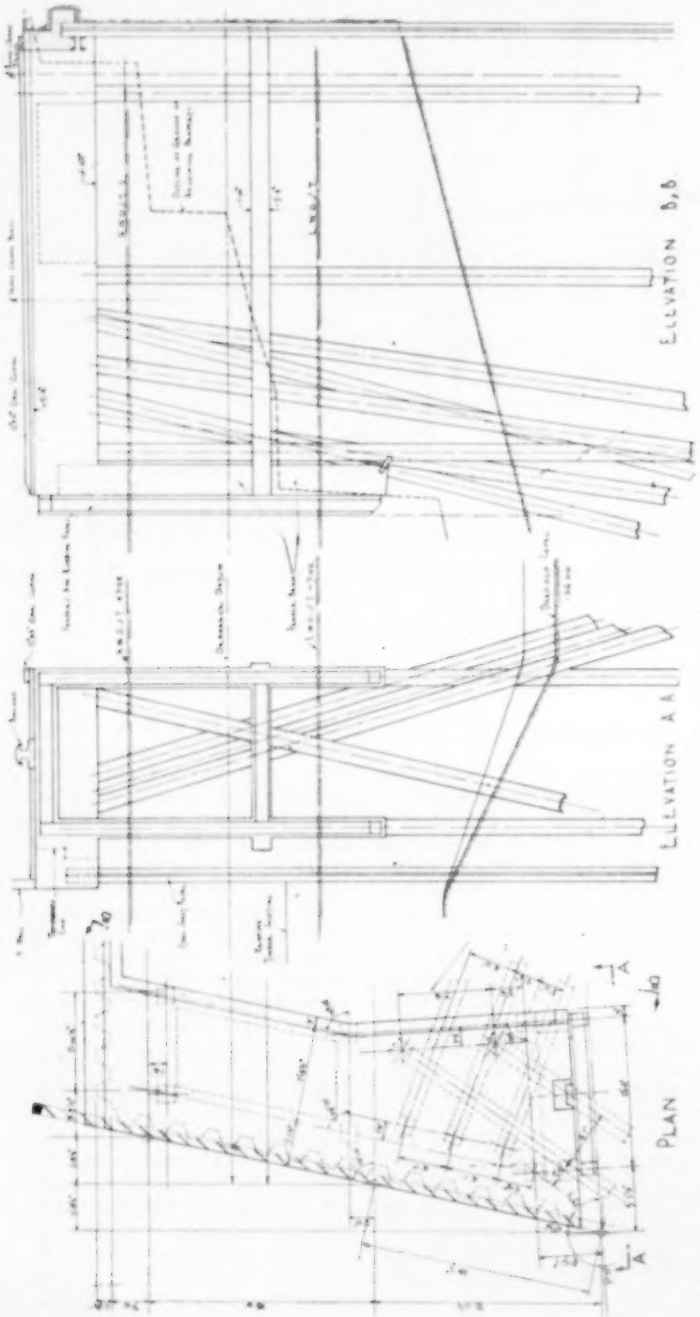


Fig. 6.—Bollard Quay.

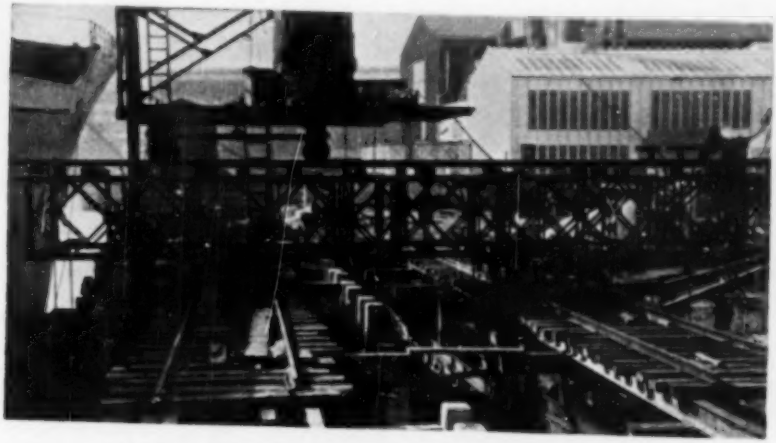


Fig. 7.—Pile-driver on Cantilevered Undercarriage.

tering for these beams, which carried the wet concrete and also supported the ends of the precast braces and the precast fender-beams, was therefore suspended by long tie-rods from steel channels bolted to the piles at the upper lifting-holes (*Fig.* 8). The deck is shown in course of construction in *Fig.* 9.

An unforeseen hazard was that newly-launched hulls of ships of up to 10,000 tons were, while being fitted out, moored to the old quay at the middle of the new work. During high winds these ships swung close to the end of the cantilevered truss, and the heavy mooring wire-ropes passed across the new work and rose and fell

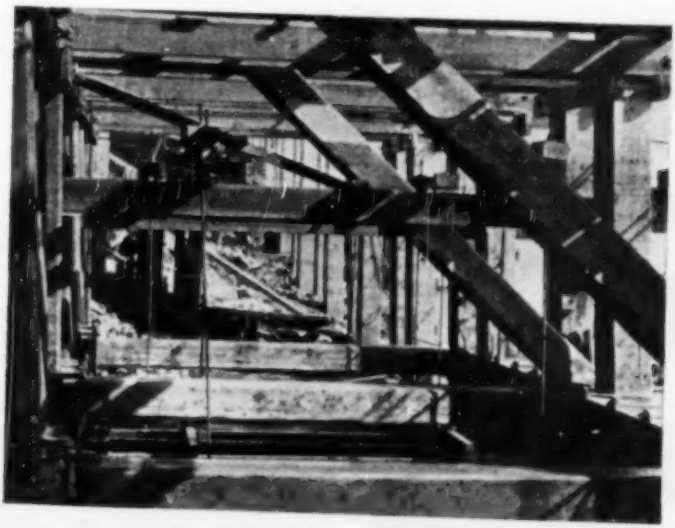


Fig. 8.—Suspended Shuttering.

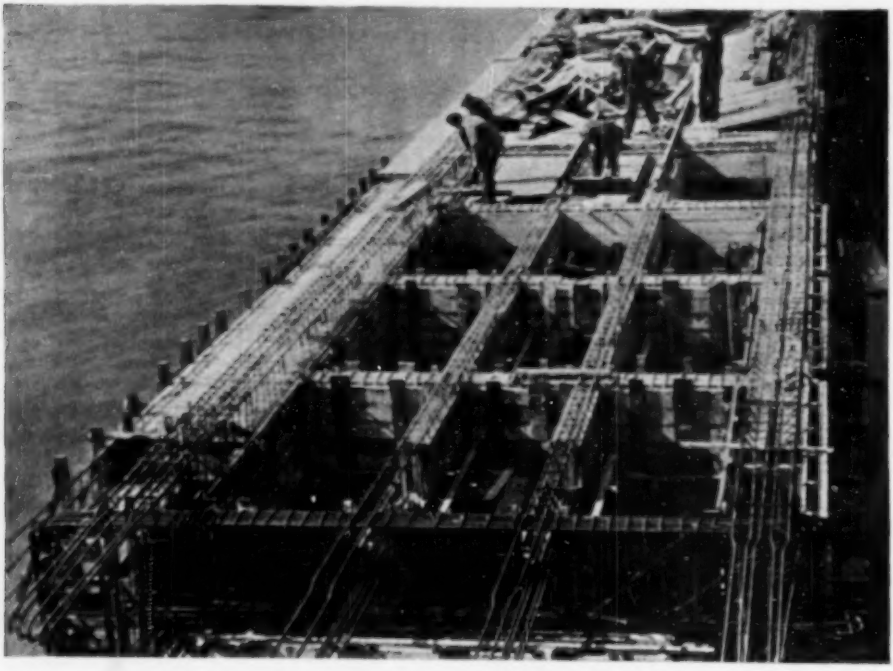


Fig. 9.—Construction of Deck.

with damaging and dangerous effect upon the temporary timbering and shuttering and upon the travelling derrick.

The consulting engineers for the new works are Messrs. R. T. James & Partners, and the contractors are Messrs. George Wimpey & Co., Ltd. The original contract price was about £100,000.

Competition for the Design of Bridges.

In view of the Special Roads Act of 1949, which authorises the construction of motorways in Great Britain, the Cement and Concrete Association recently sponsored a competition for bridge designs. The competition was open to engineers and architects normally resident in Great Britain and Northern Ireland. Prizes of £500, £300, and £200 were offered. Competitors were required to submit one design for each of three types of bridge suitable for carrying a road or footpath over a motorway.

The assessors were Sir Percy Thomas, O.B.E. (Past President of the Royal Institute of British Architects); Mr. J. Cuerel (Partner, Messrs. Rendel, Palmer and Tritton); Mr. A. Moller (Director, Peter Lind & Co., Ltd.); Mr. E. John Powell (Engineer and Surveyor, County of Glamorgan); and Mr. J. Reed (Partner, Messrs. Blyth & Blyth).

In the opinion of the assessors, the designs submitted by Atkins & Partners and Clive Pascall, and by Lindsey Drake, Denys Lasdun, A. J. Harris, and W. M. Johns, were so equal in merit that they have placed them together and divided the first and second prizes between them. The third prize is awarded to Louis Erdi and A. J. B. Birkmyer.

"Vacuum" Concrete.

THE following notes on cast-in-situ concrete treated by vacuum processes have been contributed by Mr. K. P. Billner, of New York, who was a pioneer of this process.

The basic vacuum process is a method of expelling the excess water in placed concrete by creating a high vacuum within a suction mat in contact with the concrete. With a suction equivalent to 23 in. of mercury, a pressure of 1500 lb. per square foot due to atmospheric pressure on the outside of the mat forces the mat against the concrete, expels the water, and in large measure closes the water voids. As a result a very dry concrete is obtained almost immediately, although the concrete when placed may have a comparatively high water content to produce the desired degree of workability.

For cast-in-situ construction, small and easily-handled vacuum mats are used. When enough concrete has been placed to permit the finishers to start screeding, suction mats about 3 ft. by 4 ft. are laid side by side on the freshly screeded concrete to form a continuous band 4 ft. wide across the concrete. As each mat is placed it is connected by a 1-in. vacuum hose to an outlet on the manifold which is connected to a pump. Suction is applied or cut off by a valve on the back of each mat. Suction is applied to each mat as it is placed. The speed of the work is generally adjusted so that by the time the last mat is placed it is time to shut off the suction on the first mat. By ropes attached to the back of the mat, each mat in turn is moved ahead a distance of 4 ft. (the length of the mat) until the entire line of mats is advanced 4 ft. This cycle is repeated throughout the working day.

Processing for five to ten minutes may make a 6-in. slab of concrete of ordinary workability so hard that a man can stand on it without making an appreciable im-

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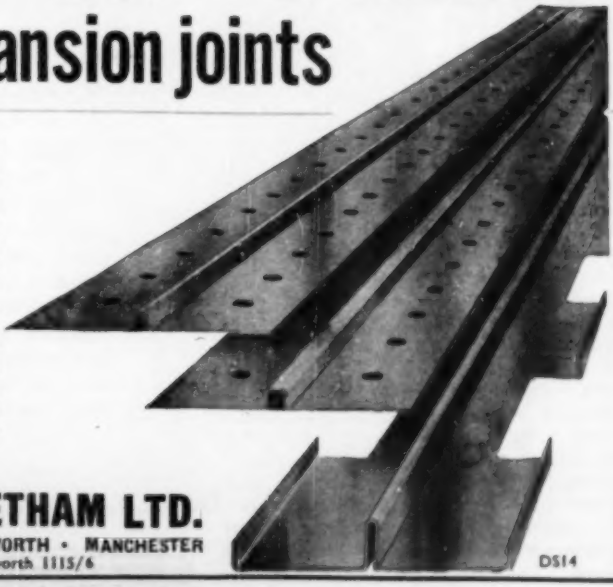

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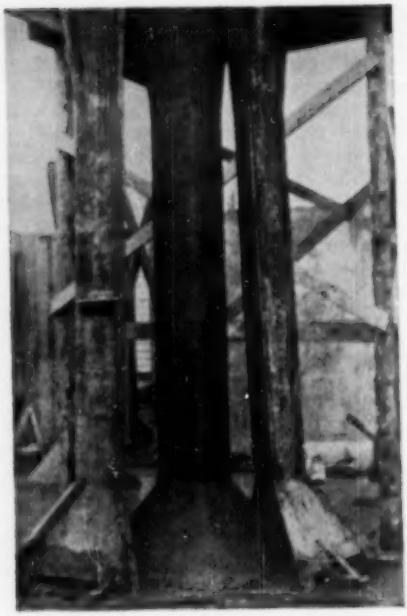

Fig. 1.—Vacuum-lined Column Shutters.

print. The finishers can float and trowel the slab ten to fifteen minutes after the concrete has been screeded. Two men with one set of mats can treat 10,000 sq. ft. of slab per day.

For dams and culverts the vacuum mat is applied as a lining to ordinary shuttering. By interposing a series of horizontal seals or cut-offs in the vacuum-mat construction, the vacuum process is applied in a series of shallow lifts making possible the use of lighter shutters. In an 8-ft. wall, horizontal seals might be at every 2 ft. of the height. As the shutter is filled, suction is applied as soon as the first seal is covered, and so on with each succeeding 2-ft. lift. As the process quickly changes the concrete from a fluid to a more nearly solid mass, the liquid pressure on the shuttering ceases and the shuttering need be designed for a head of only 3 ft. instead of 8 ft. The concrete is hard enough within an hour after the last concrete is placed for the wall shutters to be removed.

A reservoir recently completed at Brussels was constructed by the vacuum process. The structure is 100 ft. by 120 ft. by 12 ft. high. The column footings and the floor were cast first. Then eighty 12-in. square columns, flared to 3 ft. square at the bottom and to 18 in. square at the top, were cast, using two vacuum-lined steel moulds (*Fig.* 1) which were held to the floor by suction. The concrete was consolidated by a small vibrator fixed to the moulds, also by suction. The moulds were released half-an-hour after the concrete was placed and were used five times each day. The outer walls, which are 12 ft. high, were cast similarly. The roof slabs are ribbed, and were precast in concrete moulds, vacuum processed, and then lifted and set in place on the columns. The roof is designed for a load of 500 lb. per square foot because it carries 2 ft. 6 in. of earth as well as the superimposed load. The precast roof slabs were prestressed by the Belgian method.

The Economic Design of Rectangular Reinforced Concrete Sections

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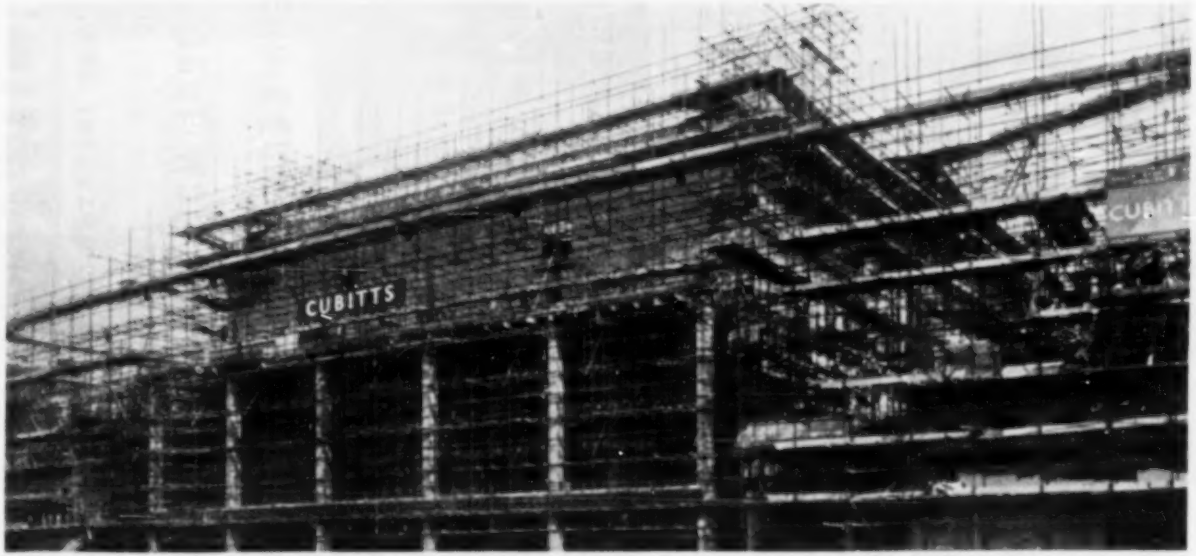

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A Reinforced Concrete Bridge at Heidelberg.

THE reconstructed Friedrichs bridge (*Fig.* 1) at Heidelberg, which is described in a recent number of "Die Bautechnik", has some unusual features. The superstructure was destroyed during the war. The new superstructure is carried on the old foundations, which have been strengthened and enlarged. The intermediate piers are uncommon, as each has a double cantilever at the head which permits spans of 126 ft. to be bridged with members the constructional depth of which is only 7 ft. 3 in. A single cantilever is provided on the end piers. The width of the intermediate piers is 13 ft., and the cantilevered-head is 47 ft. 7 in. wide. Consequently the span of the suspended beam is only 78 ft. 9 in. A comparison with other designs showed that this method was the most economical in steel. The negative bending moment is reduced by the wide piers, and, by selecting a suitable position for the point of contraflexure, the negative bending moment at the pier and the positive bending moment at midspan of the suspended beam are equal.

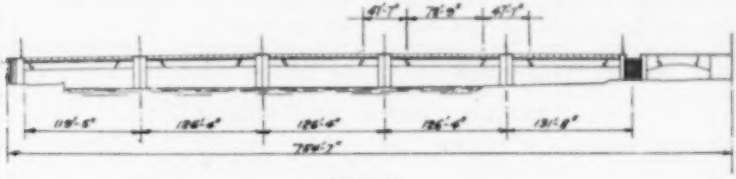

Fig. 1.

The beams were cast in situ in steel

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shuttering carried on temporary suspended steel girders. An initial deflection was caused by loading the steel girders with ballast-boxes containing water after the shuttering was erected and the reinforcement was in place. The loads were applied at two points so as to produce deflections equal to the deflections due to the weight of the beam. While the concrete was being placed, water was released from the boxes to reduce the load in proportion to the increasing weight of concrete, so that the level of the beam remained constant. There are nine longitudinal tee-beams. The cantilevered heads of the piers are cellular.

A constructional disadvantage of this design is the number of bearings required, as there are 108 in the entire structure. Six expansion joints permitting movement of railway track are also provided.

Lecture Courses on Concrete.

THE following courses of lectures are announced:

London County Council School of Building, Ferndale Road, London, S.W.4.—Dr. Paul Abeles will give eleven weekly lectures, starting January 10, 1951, on prestressed concrete. Mr. W. P. L. Eagles is lecturing on Tuesday and Thursday evenings throughout the session on shuttering.

Cardiff Technical College, Crury's Road Annexe, Cardiff.—Mr. W. M. Colly will give nine lectures, starting on January 10, 1951, on concrete technology.

Hammersmith School of Building, Lime Grove, London, W.12.—Mr. R. H. Elvery will give six weekly lectures, starting on January 30, 1951, on concrete technology.

The Polytechnic, Regent Street, London, W.1.—Mr. F. W. Gifford will give six lectures, starting on February 9, 1951, on prestressed concrete.

Battersea Polytechnic, London, S.W.11.—Mr. A. Goldstein will give six lectures, starting on February 13, 1951, on prestressed concrete.

Further information about these courses may be had from the addresses given.

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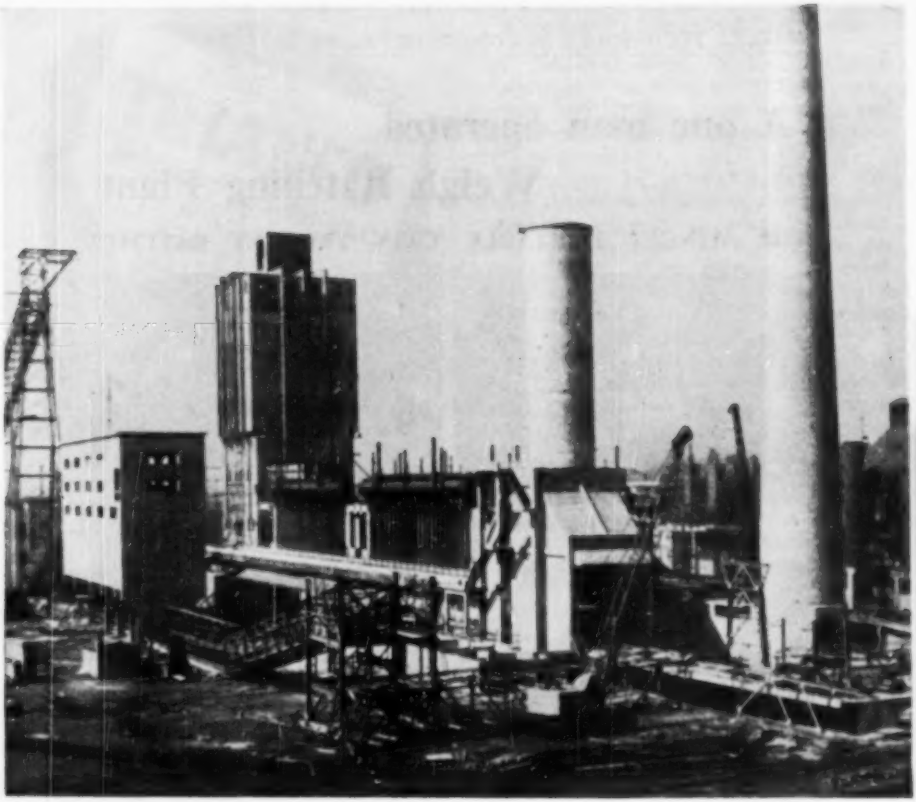

A general view of the new coking plant at the East Greenwich Gasworks, designed and constructed by Messrs. Simon-Carves, Ltd., for the South Eastern Gas Board. All the structures shown are supported on Simplex Concrete Cast-in-Situ Piles.

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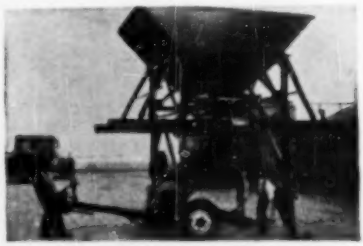

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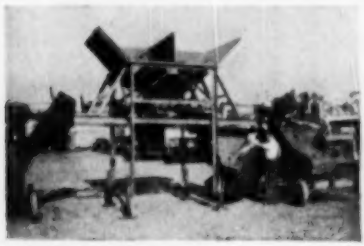

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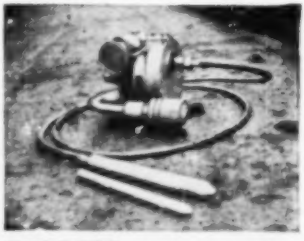

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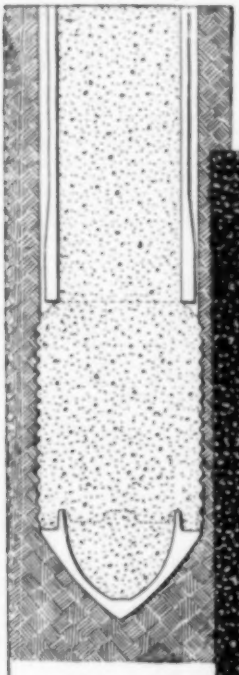

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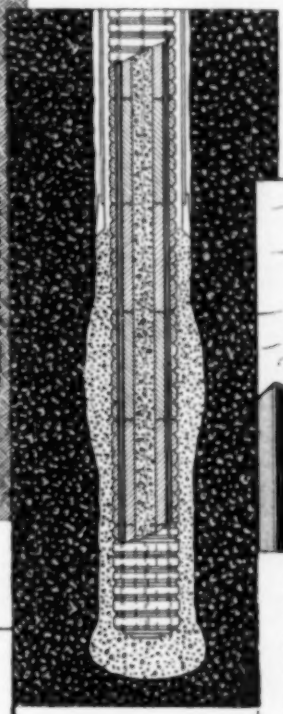

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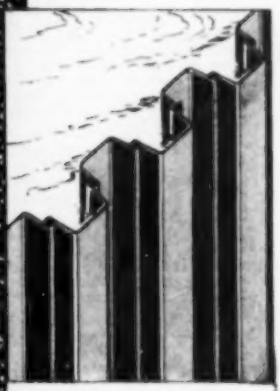

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